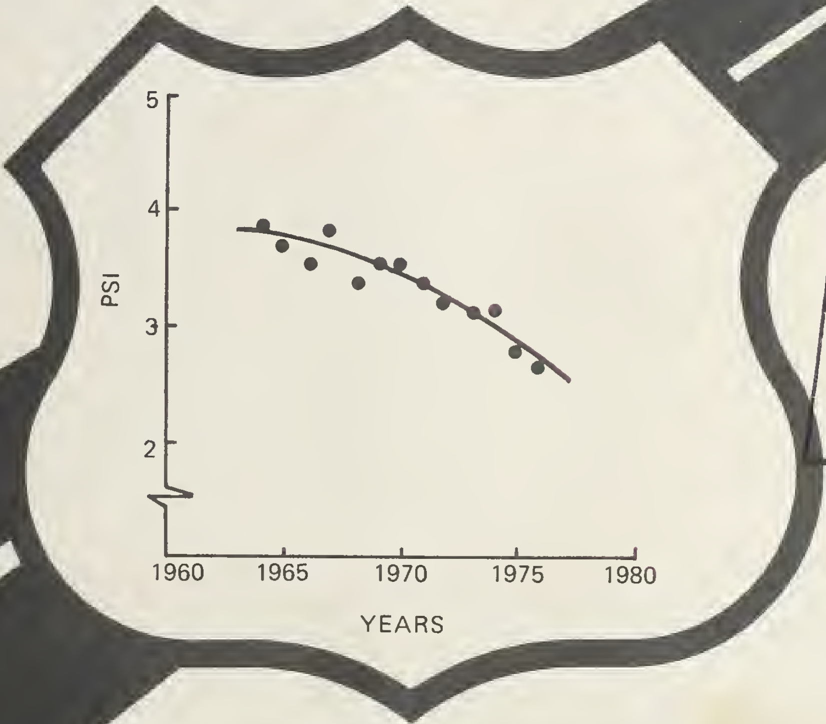
rt No. FHWA/RD-80/098

TING PAVEMENT DISTRESS TO SERVICEABILITY AND PERFORMANCE

February 1981
Final Report



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Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Structures and Applied Mechanics Division
Washington, D.C. 20590

FOREWORD

This report presents the results of research conducted by Austin Research Engineers, Inc., for the Federal Highway Administration (FHWA), Office of Research, under contract DOT-FH-11-9393. This research study was part of FCP Project 5D, "Structural Rehabilitation of Pavement Systems." Regression Analyses, Markov Processes, Bayesian Analysis, and Utility Theory were examined as analytical tools useful in relating distress to performance. Available field data on all types of pavement distress were correlated with several forms of pavement performance to try to define meaningful relationships. Several specific performance models of limited applicability are reported, along with some generally applicable techniques and recommended methods.

Copies of the report are being distributed by memorandum to individual researchers. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Richard E. Hay / for
Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

Technical Report Documentation Page

1. Report No. FHWA/RD-80/098	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle RELATING PAVEMENT DISTRESS TO SERVICEABILITY AND PERFORMANCE		5. Report Date February 1981
		6. Performing Organization Code
7. Author(s) W.R. Hudson, F.N. Finn, R.D. Pedigo, F.L. Roberts		8. Performing Organization Report No.
9. Performing Organization Name and Address Austin Research Engineers Inc. 2600 Dellana Lane Austin, Texas 78746		10. Work Unit No. (TRAIS) FCP 35D1-282
		11. Contract or Grant No. DOT-FH-11-9393
12. Sponsoring Agency Name and Address Federal Highway Administration Offices of Research and Development Washington, D. C. 20590		13. Type of Report and Period Covered Final Report
		14. Sponsoring Agency Code
15. Supplementary Notes Mr. Richard A. McComb FHWA Contract Manager HRS-14		

16. Abstract

In 1970 an FHWA Workshop defined major problems relating to improved pavements. The number one problem cited was the problem of relating pavement distress to pavement performance in an understandable predictable way. This report examines available data on all types of pavement distress and on several forms of pavement performance to define meaningful relationships. Regression analyses, Markov Processes, Bayesian Analysis, and Utility theory are examined as analytical tools useful in relating distress to performance. Several specific performance models of limited applicability are reported, along with some generally applicable techniques and recommended methods.

This study shows that useful relationships can be obtained from existing data and are, in fact, being used in several states and other highway agencies. In each case, however, it is shown that significant improvements in the distress-performance relationships are needed which can only be obtained through the conscientious observation of distress and performance of pavements under a variety of environmental and load conditions for a reasonable period of time. e.g. (approximately ten years). Several suggestions and recommendations for improvement are included in the study.

17. Key Words pavement, performance, pavement distress, pavement models, pavement management	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Va. 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
UNCLASSIFIED	UNCLASSIFIED	271	

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TABLE OF CONTENTS

	Page
LIST OF FIGURES	vii
LIST OF TABLES	xii
CHAPTER 1. INTRODUCTION	1
DISTRESS/PERFORMANCE PROBLEM	1
Distress Development	4
Current Performance Methodologies	5
OBJECTIVES	8
SCOPE	8
RESEARCH APPROACH	8
Advisory Panel Review	9
Catalogue Pavement Distress	9
Define Typical Distress and Serviceability Performance Development Patterns	10
Evaluation and Implementation of Methodologies	10
REPORT ORGANIZATION	12
CHAPTER 2. PAVEMENT DISTRESS AND PERFORMANCE MEASURES	13
DEFINITION OF TERMS	13
PAVEMENT DISTRESS	14
Comprehensive Distress Listings	14
Selection of Distress Types for Further Study	15
PAVEMENT SERVICEABILITY AND PERFORMANCE	18
Pavement Serviceability	18
Pavement Performance	20
EVALUATION OF EXISTING MECHANISTIC DISTRESS MODELS	21

TABLE OF CONTENTS (continued)

Page

CHAPTER 3. DATA COLLECTION 24

SPECIAL RESEARCH STUDIES 24

AASHO Road Test 24
Brampton Test Road 30
TTI Data Base 30

STATE TRANSPORTATION AGENCIES 30

Washington 30
Utah 39
Minnesota 39
Illinois 46
Georgia 46

PUBLISHED REPORTS 54

CHAPTER 4. GENERAL DISTRESS TO PERFORMANCE RELATIONSHIPS 55

BASIC SEQUENCE OF PAVEMENT DISTRESS DEVELOPMENT 55

Secondary Distresses 58

DISTRESS AND SERVICEABILITY PATTERNS 61

Flexible Pavement 61
Rigid Pavements 67

CHAPTER 5. DATA ANALYSIS AND MODELING 72

VERIFICATION OF DISTRESS AND PERFORMANCE PATTERNS 72

Flexible Pavement Patterns 72
Rigid Pavement Patterns 75

TIME INDEPENDENT DISTRESS/SERVICEABILITY RELATIONSHIPS . . . 76

Flexible Pavement Models 76
Rigid Pavement Models 92

TIME DEPENDENT DISTRESS/SERVICEABILITY RELATIONSHIPS 113

TABLE OF CONTENTS (continued)

Page

Flexible Pavements 115
Rigid Pavements 124

CHAPTER 6. RELATED FINDINGS 135

DATA REQUIREMENTS FOR PERFORMANCE MODELING 135

"Ideal" Experiment Design 138
Analyses 143
Other Design Approaches 145

PERFORMANCE MODEL REQUIREMENTS FOR PAVEMENT MANAGEMENT . . . 145

STOCHASTIC SERVICEABILITY DETERIORATION MODEL FOR RIGID PAVEMENTS 147

TOWARD AN OVERALL MEASURE OF PAVEMENT PERFORMANCE 157

Utility Theory 158
Unique Sums 160

CHAPTER 7. RECOMMENDATIONS 164

SIGNIFICANT PROBLEMS ENCOUNTERED 164

Pavement Condition Variables 165
Time Effects 166
Serviceability or Roughness Measurements and Calibration 166
Overall Performance Concepts 168

RECOMMENDATIONS 169

Research Problem Statement 1. Pavement Performance Models Using Combined Data from Accelerated Pavement Testing and Long-Term Field Performance Observations 169
Research Problem Statement 2. Long-Term Pavement Observations for Pavement Management Systems 170
Problem Statement 3. Improved Serviceability Measurements and Roughness Calibration Technique . . 171
Problem Statement 4. Coordination of Activities Over a Range of Environment, Load, Materials, and Other Factors 172

TABLE OF CONTENTS (continued)

Page

SUMMARY 173

APPENDIX A. DEFINITION OF TERMS 175

APPENDIX B. SELECTION OF PAVEMENT DISTRESS TYPES FOR FURTHER STUDY 178

APPENDIX C. EVALUATION OF DISTRESS MODELS 201

FLEXIBLE PAVEMENT 201

Rutting 202
Fatigue 206
Low Temperature Cracking 212

JOINTED CONCRETE PAVEMENT 213

Serviceability Performance Analysis 213
Fatigue Damage Analysis 215

CONTINUOUSLY REINFORCED CONCRETE PAVEMENT 220

Fatigue Analysis 222
Low Temperature and Shrinkage Cracking 223

JOINTED REINFORCED CONCRETE PAVEMENT 226

Low Temperature and Shrinkage Cracking 227

SUMMARY OF MODEL STUDY RESULTS 230

APPENDIX D. EXPANDED SEQUENCE OF DEVELOPMENT FOR SELECTED DISTRESSES 231

Flexible Pavement Distress 231
Composite Pavement Distress 231
Rigid Pavement Distress 231

REFERENCES 249

LIST OF FIGURES

Figure No.		Page
1	Schematic sketch of scope of the research study on relating pavement distress to performance	2
2	Simplified block diagram of the pavement system (after Ref. 20)	3
3	Potential distress and serviceability patterns	11
4	Example flexible pavement performance record AASHO Road Test	25
5	Example rigid pavement performance record, AASHO Road Test	26
6	Example distress index (DI) plot, Brampton Test Road	31
7	Example rut depth plot, Brampton Test Road	32
8	Example riding comfort rating (RCR) plot, Brampton Test Road	33
9	Example flexible pavement data record, TTI Data Base	36
10	Example composite pavement data record, TTI Data Base	37
11	Example pavement condition history, Utah	43
12	Example pavement performance summary, Utah	44
13	Example summary sheet, Minnesota	47
14	Sample summary sheet, Rehabilitated AASHO Test Road, Illinois	50
15	Basic distress sequence	56
16	Development of fatigue cracking in an asphalt concrete pavement	57
17	Increased fatigue cracking (secondary distress) in an asphalt concrete pavement due to moisture infiltration	59
18	Increased fatigue cracking (secondary distress) in an asphalt concrete pavement due to pumping erosion	60

LIST OF FIGURES (continued)

Figure No.		Page
19	Distress and serviceability history patterns, flexible pavements	62
20	Flexible pavement history	63
21	Flexible pavement distress/serviceability history with rutting as the primary distress	65
22	Flexible pavement distress/serviceability history with fatigue cracking as the primary distress	66
23	Distress and serviceability history patterns, rigid pavements	68
24	Rigid pavement history, CRCP	69
25	Primary pattern of serviceability history, Minnesota	74
26	"PSR" as a function of rutting, Minnesota	77
27	"PSI" as a function of rutting, Minnesota	78
28	"PSR" versus "PSI", Minnesota	80
29	"PSR" versus rut depth for those sections with C + P > 50, Minnesota	81
30	Riding comfort versus rut depth, Brampton	82
31	Bump count levels for alligator cracking of severity 1 and extent 1, Washington	84
32	Serviceability and transverse cracking history, rehabilitated AASHO Test Road section 076	93
33	Serviceability and transverse cracking history, rehabilitated AASHO Test Road section 404	94
34	Serviceability and transverse cracking history, rehabilitated AASHO Test Road section 338	95
35	PSR as a function of transverse cracking, CRC pavement	97
36	Change in serviceability versus change in major cracking for selected rigid pavement section, AASHO Road Test	99

LIST OF FIGURES (continued)

Figure No.		Page
37	Serviceability related to "D" cracking, rehabilitated AASHO Test Road	101
38	PSI versus joint faulting for pavement with average faulting exceeding 0.16 inches, rehabilitated AASHO Test Road	103
39	PSR versus joint faulting for moderately or severely faulted pavements (Ref. 60)	105
40	Serviceability versus major spalling, rehabilitated AASHO Test Road	107
41	Rutting as a function of time, Minnesota section 14	116
42	Cracking plus patching (C + P) as a function of time, Minnesota section 31	117
43	PSI as a function of time, Minnesota section 6	118
44	Distress and serviceability history, Texas section 704	119
45	Distress and serviceability history, Texas section 220	120
46	Distress and serviceability history, Texas section 1030	121
47	Delay between onset of cracking and terminal serviceability, AASHO Road Test loops 4, 5, 6 flexible pavement	123
48	Delay between onset of major cracking and terminal serviceability, AASHO Road Test loops 4, 5, 6, flexible pavements	125
49	Delay between onset of cracking and terminal serviceability, AASHO Road Test Loop 3, rigid pavement	126
50	Delay between onset of cracking and a serviceability of 3.0, AASHO Road Test loop 3, rigid pavement	127
51	Delay between damage index exceeding 100 and terminal serviceability, AASHO Road Test loop 3, rigid pavement	128
52	Influence of environmental factors on rigid pavement serviceability, AASHO Road Test	129

LIST OF FIGURES (continued)

Figure No.		Page
53	Riding quality versus accumulated load for severe Texas CRCP sections	130
54	Riding quality versus failures per mile for severe Texas CRCP sections	131
55	Comparison of predicted (equation 17) and observed serviceability histories, rehabilitated AASHO Test Road section 350	134
56	"Ideal" experiment design using 15 factors in 8 blocks (after Ref. 62)	141
57	Serviceability history using a normalized service life time scale, AASHO Road Test rigid pavements	149
58	Predicted serviceability history based on stochastic version of Equation 19	151
59	Relationship between utility value and cracking or serviceability for example problem	159
60	VESYS IIM pavement analysis and design system (Ref. 30)	203
61	Normal predictability density function used to predict area cracked (Ref. 30)	208
62	Predicted vs. actual equivalent 80 kN (18-kip) single axle load applications using AASHO equation and 16 years data from AASHO Road Test site (1958-1974) (Ref. 8)	216
63	Accuracy of new design Eq. 10 (Ref. 8)	217
64	Serviceability/performance zero-maintenance design chart for plain jointed concrete pavement (Ref. 8)	218
65	Effect of fatigue "damage" on cracking of PCC plain jointed concrete pavements (Ref. 8)	221
66	Continuously reinforced concrete pavement geometric model (Ref. 36)	224
67	Behavior of a reinforced slab subjected to temperature drop (Ref. 36)	225

LIST OF FIGURES (continued)

Figure No.		Page
68	Crack occurrence in a JRC slab (Ref. 38)	228
69	Fatigue cracking development in an asphalt concrete pavement structure (after Ref. 59)	232
70	Rutting development in an asphalt concrete pavement structure (after Ref. 59)	234
71	Low-temperature cracking development in an asphalt concrete pavement structure (after Ref. 59)	235
72	Reflection cracking development in a composite pavement structure (after Ref. 59)	236
73	Fatigue cracking development in a composite pavement structure (after Ref. 59)	237
74	Rutting development in a composite pavement structure (after Ref. 59)	239
75	Low-temperature and shrinkage cracking development in a PCC pavement structure	241
76	Development of joint and crack surface spalling in JCP and JRCP structures	242
77	Development of crack spalling in CRCP structures	243
78	Development of faulting at a joint or crack in a PCC pavement structure	244
79	Fatigue cracking development in a PCC pavement structure .	245
80	Development of D-cracking in a PCC pavement structure . .	247
81	Development of punchouts in CRCP structures	248

LIST OF TABLES

Table No.		Page
1	Priority ranking of significant distresses selected for further study (Ref. 23)	16
2	Priority ranking of significant distresses selected	19
3	Summary of models	22
4	Example AASHO Road Test data file	27
5	Flexible pavement variables, AASHO Road Test	28
6	Rigid pavement variables, AASHO Road Test	29
7	Example pavement condition record, Brampton Test Road	34
8	Flexible pavement variables, Brampton Test Road	35
9	Flexible and composite pavement variables, TTI Data Base	38
10	Example flexible pavement master file listing, Washington DOT	40
11	Example rigid pavement master file listing, Washington DOT	41
12	Flexible pavement variables, Washington DOT	42
13	Rigid pavement variables, Washington DOT	42
14	Flexible pavement variables, Utah	45
15	Portion of data record from summary sheet, Minnesota	48
16	Flexible pavement variables, Minnesota	49
17	Partial data record, rehabilitated AASHO Test Road, Illinois	51
18	Rigid pavement variables, rehabilitated AASHO Test Road, Illinois	52
19	Rigid pavement variables, Georgia	53

Table No.		Page
20	Transformed variables for multiple regression, Minnesota	85
21	Results of stepwise regression, Minnesota, PSI equation	87
22	Results of stepwise regression, Minnesota, PSR equation	88
23	Results of stepwise regression by gravel equivalent class, Minnesota, PSI equation	90
24	Results of stepwise regression by gravel equivalent class, Minnesota, PSR equation	91
25	Bump count values for moderate and severe cracking, Washington rigid pavements	96
26	Serviceability related to major cracking, AASHO Road Test rigid pavements	100
27	Bump counts by level of faulting, Washington rigid pavements	104
28	Bump counts by level of joint spalling, Washington rigid pavements	106
29	Data for 49 selected rigid pavements (Ref. 3)	109
30	Rigid pavement variables examined by Program ALLRSQ	110
31	Combinations of rigid pavement variables selected by Program ALLRSQ for use in Program FIXREG	111
32	Statistical analysis results for rigid pavement from Program FIXREG	112
33	Analysis of variance for lack of fit, Equations 11 and 12	114
34	Potential factors to be included in the design of an ideal experiment to collect data for pavement performance modeling	140

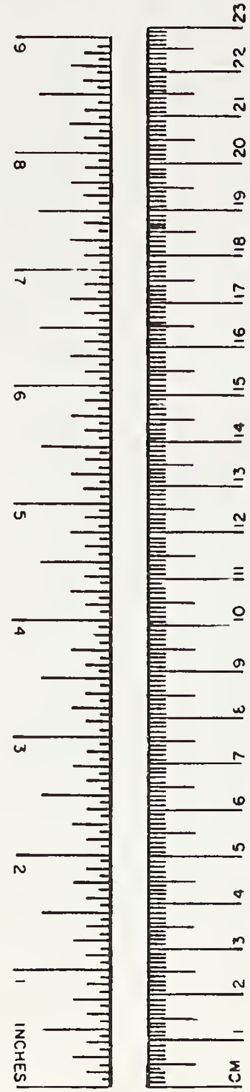
Table No.		Page
35	Additional treatment combinations needed to investigate curvature in the "ideal" experiment	142
36	List of possible pavement serviceability states for Markov model	153
37	Transition probability matrix for Markov example problem	154
38	Initial state specification for $\overline{PSI} = 4.5$, $\sigma = 0.1$	156
39	Unique sum categorization for three attributes with four levels (after Ref. 72)	162
40	Unique sum categorization for four attributes with four levels	163
41	Flexible pavement distress (Ref. 21)	179
42	Rigid pavement distress (Ref. 21)	181
43	Summary of distress types found on flexible pavements rated as moderate to severe (Ref. 22)	183
44	Summary of distress types on composite pavements where distress is rated as moderate to severe (Ref. 22)	184
45	Summary of distress types on JCP rated as moderate to severe (Ref. 22)	185
46	Summary of distress types found on JRCP rated as moderate to severe (Ref. 22)	186
47	Summary of the most predominant distress types found on CRCP rated moderate to severe (Ref. 22)	187
48	Deduct values for flexible pavements (Ref. 23)	188
49	Deduct values for rigid pavement (Ref. 23)	190
50	Weights for flexible pavement level of service (Ref. 23)	192

Table No.		Page
51	Weights for rigid pavement level of service (Ref. 23)	192
52	Pavement distress by category and pavement type	193
53	Priority ranking of significant distresses selected for further study (Ref. 23)	194
54	Flexible pavement utilities for various frequencies of rural (α_1) and segment (α_2) rehabilitation	197
55	Rigid pavement utilities for various frequencies of rural (α_1) and segment (α_2) rehabilitation	197
56	Average deduct values for flexible pavement distress	199
57	Average deduct values for rigid pavement distress	199

METRIC CONVERSION FACTORS

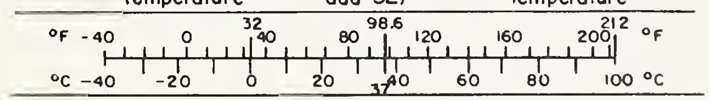
APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.6	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons(2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares(10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	8.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	36	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



CHAPTER 1. INTRODUCTION

Significant progress has been made since 1966 in improving theoretical and empirical models for predicting pavement behavior. The AASHO Road Test provided significant empirical data for relating axle loads, pavement structural properties and performance as defined by number of axle loads carried. It has been difficult, however, based on available information to quantify improved models for predicting pavement performance. Most pavement models, particularly those with adequate theoretical bases, are concerned with predicting pavement behavior as defined by mechanistic values of stress and strain or deformation. Pavement performance, on the other hand, while certainly dependent upon stress and strain is far more complicated and is a result of complex interactions of cracking, permanent deformation, faulting disintegration, and other physical distress manifestations.

Significant other research has been carried out in the definition of improved theoretical models for predicting behavior and distress. This study and this report concerns itself primarily with the relationship of distress to performance and assumes that models for predicting behavior and distress are available from other sources. Figure 1, first developed by a Committee of the Transportation Research Board, depicts graphically the concern of this research study.

In their paper on a generalized framework for pavement rehabilitation, Hudson and Finn (Ref. 20)[1] present a simplified version of the pavement management problem which illustrates the relationship of behavior, distress, and performance (Fig. 2).

DISTRESS/PERFORMANCE PROBLEM

The problem of defining pavement failure has never been adequately solved. This lack of definition came into sharp focus at the WASHO Road Test as disagreement occurred when the staff and advisory committee tried to define the point at which a test section had "failed". One reason for these differences occurred as a result of the divergent viewpoints from which each of the engineers and users viewed pavement failure. Pavement failure is perceived differently by the construction engineer, the design engineer, and the maintenance engineer. When failure is based on subjectively derived criteria, differences in opinion between any two individuals or groups of engineers can vary significantly. In order to resolve the problem of defining failure, an alternative approach called the serviceability/performance concept was developed by

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[1]Hudson, W.R. and F.N. Finn, "A General Framework for Pavement Rehabilitation", Report No. FHWA-RD-74-60, June 1974.

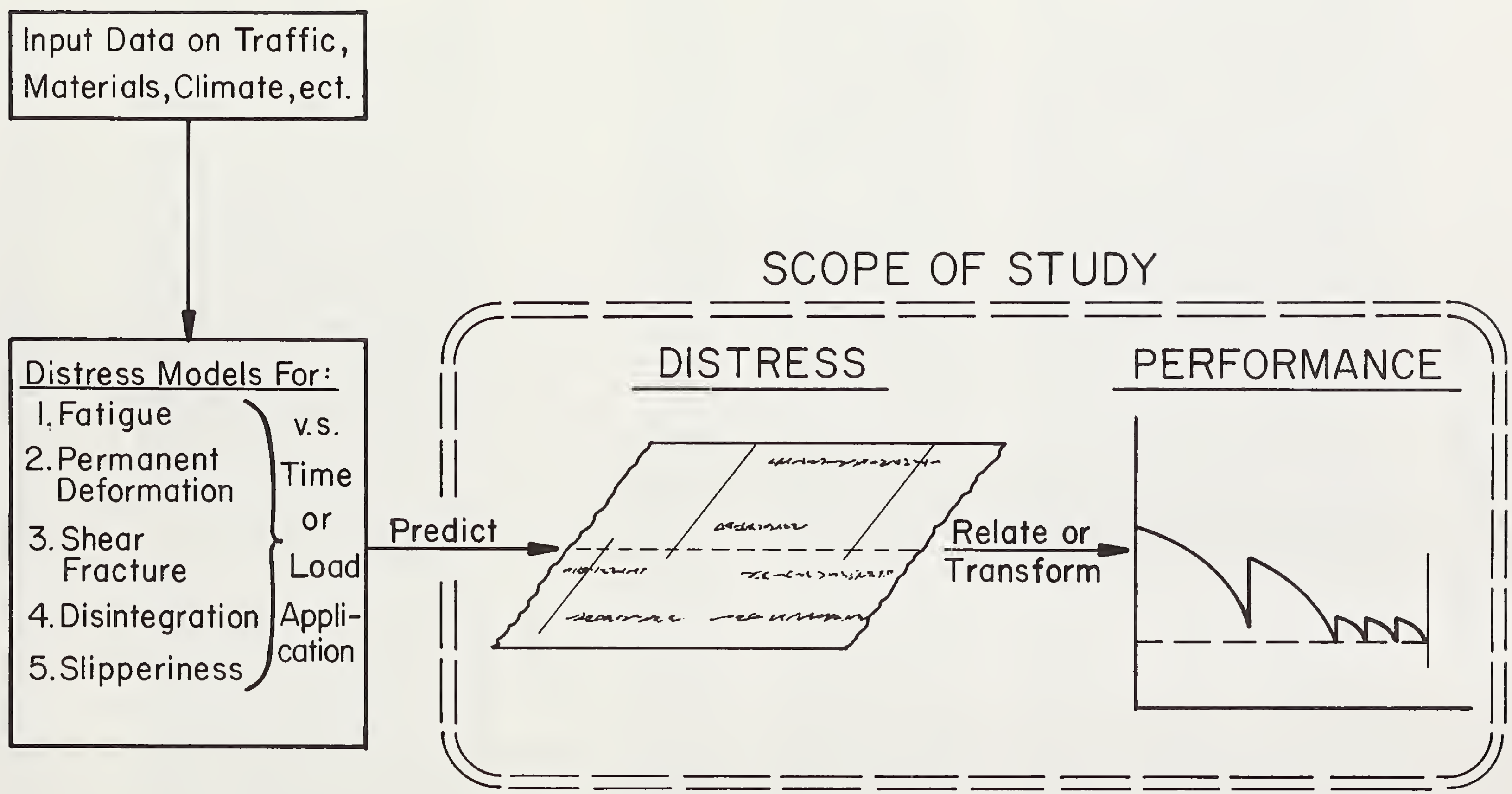

Figure 1 Schematic sketch of scope of the research study on relating pavement distress to performance

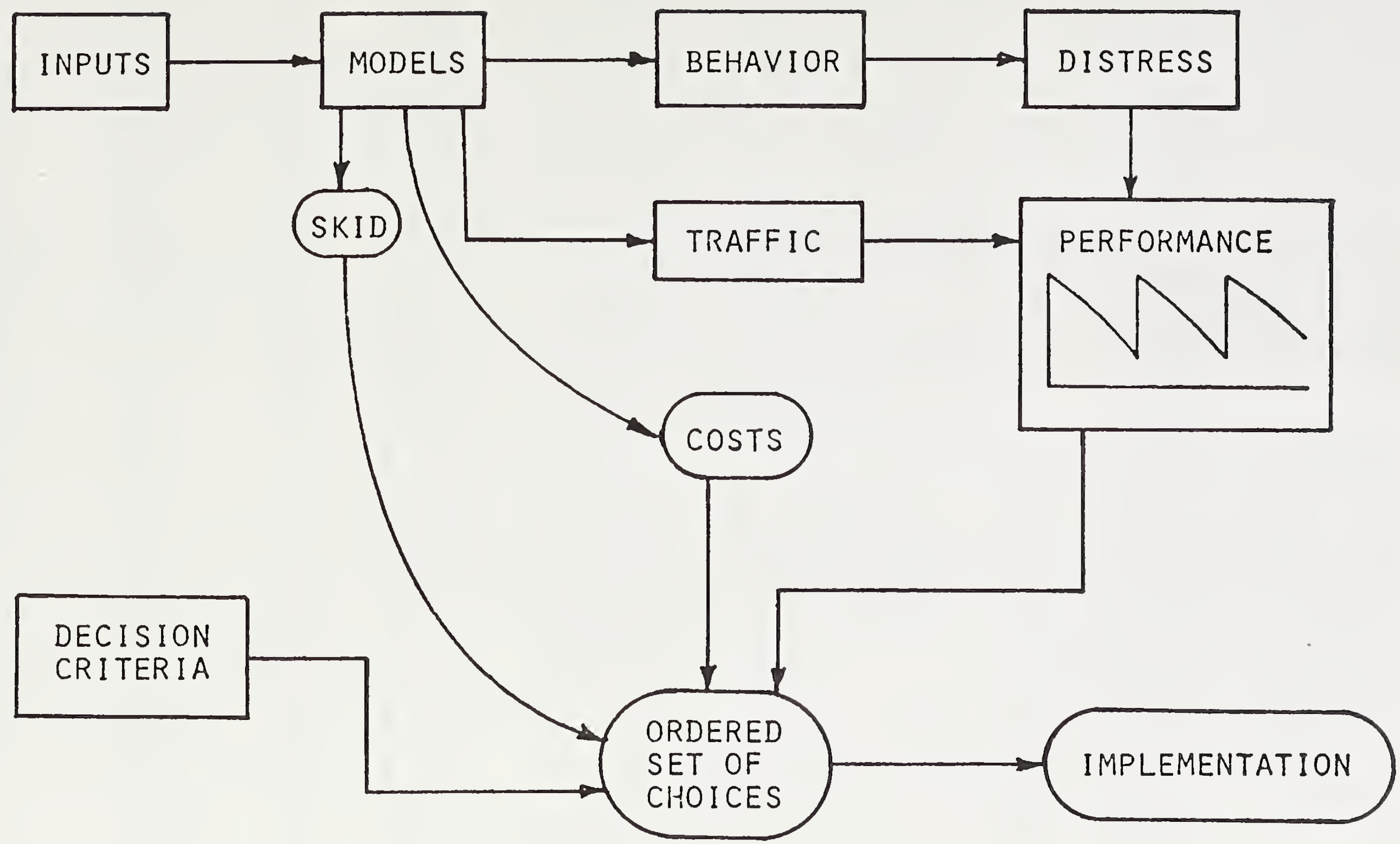

Figure 2 Simplified Block Diagram of the Pavement System (after Ref. 20)

Carey and Irick (Ref. 1)[1] to describe the condition or usefulness of a pavement. In this approach, serviceability is defined as the ability of a pavement to serve its function at a specific time. The function of the pavement is defined as the ability to carry high speed, high volume traffic at an acceptable riding quality. In recent years, the requirement for adequate safety has been added to the criteria. Thus, the function of a pavement is to carry the traffic safely and comfortably at design speed. Performance is the integration of the serviceability history. "Failure", then, can be defined as an unacceptable serviceability level. At the AASHO Road Test, this was prescribed to be when the pavement reached a terminal serviceability level of 1.5.

Distress Development

As traffic loads, environment, and other forces act upon the pavement system, the pavement responds with stress, strain, deformation, fracture, polishing, or wear. These responses to load and wear are termed behavior. Most of the analysis models currently used to design pavement structures are, in fact, predictors of behavior. When the predicted behavior reaches a limiting response value, distress results. These distresses accumulate until serviceability declines. This serviceability loss can occur as a result of the accumulation of a single type of distress or as a combination of several types. Certain types of distress, such as a single crack, do not in themselves cause pavement failure. However, as distress accumulates, they may combine with other types of distress such as deformation and disintegration to cause the level of serviceability to drop below an acceptable level with the result that "failure" is said to occur. Since the performance of a pavement has been tied to its ability to fulfill its design function, the accumulation of distress acts to destroy that ability to serve traffic. The accumulation of distress and the relation between distress and performance is then very complex, having at least three components: (1) primary, (2) secondary, and (3) time dependent.

Primary distresses include those which have a direct and immediate effect on serviceability/performance. Secondary distresses are those which occur as a corollary of certain types of primary distresses. For example, the cracking of a pavement due to temperature is a primary distress. The crack itself may not reduce serviceability, however, entry of water into the crack may precipitate a weakening of the underlying pavement materials which may produce permanent deformations that lead to a loss in serviceability. These deformations are a secondary distress.

The time dependent distress may have no immediate effect on serviceability but given adequate time will lead to a reduction in the service-

[1]Carey, W.N. and P.E. Irick, "The Serviceability-Performance Concept", Bulletin 250, Highway Research Board, 1960.

ability of the pavement structure. For example, transverse cracks in continuously reinforced concrete pavement are designed and expected. However, when the crack spacing becomes too close, the accumulation of load applications and thus time may produce deep spalling or punchouts at these closely spaced cracks; thus, distress accumulates.

In the last decade the development of pavement management systems has brought to the forefront a need for development of objective functions to be used in the pavement management system for optimization of design, rehabilitation, and maintenance. This objective function must include pavement performance. Currently, the relationship must be empirical in nature because of the lack of definitive information that can be used to develop an objective function based on real data. In order for rational pavement management systems to be developed, this missing objective function must be obtained.

Current Performance Prediction Methodologies

In order to develop relationships, several methodologies have been investigated as a part of this research effort. A review of the basic methodologies reported in the literature shows several of the techniques currently being used to define relationships or to serve as surrogate performance functions as discussed in the following sections.

Multiple Regression Analyses. The multiple regression analysis approach can be used to quantify pavement performance as a function of those physical factors known to engineers to significantly influence the deterioration of a pavement. Some attempts have been made in this regard (Ref. 2[1],3[2],4[3]).

Kulkarni, et al (Ref. 4) have examined the use of the multiple regression approach for the Washington Department of Highways to relate pavement "rating" to load, material, structural, and environmental variables. Because of insufficient information, they were not successful in developing an acceptable relationship between rating number and time, but it was concluded that a continuing effort to use regression analyses

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[1]Darter, M.I. and E.J. Barenberg, "Zero Maintenance: Results of Field Studies on the Performance Requirements and Capabilities of Conventional Pavement Systems", Report prepared for FHWA, April 1975.

[2]"The AASHO Road Test Report 5, Pavement Research", Highway Research Board Special Report 61E, Publication No. 954, National Academy of Sciences - National Research Council, 1962.

[3]Kulkarni, R., R. LeClerc, F.N. Finn, and H. Sandahl, "Development of a Pavement Management System", Paper presented at 1976 Annual Meeting of Transportation Research Board, January 1976.

was justified. Finn (Ref. 5)[1] has successfully applied such techniques for the Arizona Department of Transportation to relate change in serviceability with time to environment, traffic, structural condition, and pavement age.

Darter, et al (Ref. 2) studied some regression models, relating serviceability to distress from References 3 and 6[2]. The AASHO Road Test staff also reported on studies relating distress to slope variance. Both studies indicated that a fair correlation with cracking is possible for PCC pavements. Faulting could improve the relationship somewhat (Ref. 2). However, the relationship between distress and slope variance was poorly defined for asphalt pavements at the AASHO Road Test. The strongest correlation is obtained with rut depth variance, which suggests the type of information required from field measurements.

Neither Darter, et al, nor the Road Test staff, nor Finn (unpublished analyses of Road Test data from NCHRP Project 1-10B) have investigated the secondary effects of primary distress manifestations, the time progression of primary distress, or the impact on future roughness or rate of change in roughness from field observations. Available data is simply inadequate.

Markov Process. A second approach is to make use of a Markov process to predict future pavement conditions (Ref. 4, 7[3], 8[4]). This process can be used to represent pavement response over a period of time. Essentially, when a pavement is first constructed, its condition normally will be good to excellent. After opening to traffic, the condition will gradually deteriorate in a series of transitions of pavement condition states from the initially excellent condition to less desirable conditions. This concept of a transition of pavement condition states is the technique used to model the deterioration of pavement

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[1]Finn, F.N., R. Kulkarni and J. McMorran, "Development of a Framework for a Pavement Management System", Final Report, Woodward-Clyde Consultants to Arizona Department of Transportation, August 1976.

[2]Roberts, F.L. and W.R. Hudson, "Pavement Serviceability Equations Using Surface Dynamics Profilometer", Research Report 733, Center for Highway Research, University of Texas, 1970.

[3]Smith, W. and C.L. Monismith, "A Maintenance Management System for Asphalt Pavements", Paper presented at Annual Meeting of Transportation Research Board, Washington, D.C., January 1976.

[4]Karan, M.A. and R.C.G. Haas, "Determining Investment Priorities for Urban Pavement Improvements". Proceedings, Association of Asphalt Paving Technologists, Volume 45, February 1976.

condition in the Markov approach.

Bayesian Analysis. This technique has been used in projects under NCHRP sponsorship (NCHRP-9-4A) (Ref. 88)[1] to determine its effectiveness as a means to quantify subjective experience into a data base of information to evaluate the influence of designer controlled variables on fatigue cracking; e.g., asphalt viscosity, proportion of asphalt concrete, percent asphalt, and density of base. This type of information is very useful in verifying the influence of certain designer controlled variables on distress; however, these general concepts have had limited use in this investigation.

Utility Theory. Utility theory has been used by Finn, et al (Ref. 5) in the development of a pavement management system for the Arizona DOT. This methodology provides a proven, theoretically sound procedure for summarizing or combining multiple attributes into a single parameter through the use of utility functions. The utility functions can be used to obtain the weighting factors based on subjective preferences of experienced engineers in assessing the relative importance of a variety of potentially uncorrelated factors. Thus, utility theory can provide a summary statistic for combining distress and performance but not for relating distress to performance.

The advantage of the use of utility theory is the high degree of certainty with which weighting factors can be obtained for combining distress and performance on a common scale; e.g., 0 to 1. For example, the utility for a road at a particular time may be: for fatigue cracking, 0.1; rutting, 0.2; skid resistance, 0.1; and roughness, 0.3. The combined utility is then 0.7. A second pavement could have values of 0.2, 0.2, 0.1 and 0.1, respectively for a utility of 0.6. Hence, the second pavement would be considered, overall, to be in a less desirable state at the time measurements were made. The primary disadvantage to the utility theory approach is its dependence on the subjective preferences of the persons used to determine the utility function.

It should be noted that the procedures used in establishing the utility function are very specific and thus provide a consistent and thoroughly tested methodology. It is also possible to "pool" utility functions from subjective preferences of several individuals in such a way as to obtain a consensus representation of those persons included in the interview process. One potential advantage, not previously mentioned, is the fact that a large data base of information is not required to establish the utility function, since it is obtained through a series of interviews. However, in order to test the reliability of the devel-

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[1]Smith, W.S., F. Finn, R. Kulkarni, C. Saraf, and K. Nair, "Bayesian Methodology for Verifying Recommendations to Minimize Asphalt Pavement Distress", NCHRP Report 213, June 1979.

oped utilities, measurements must be made, the distresses recorded and compared, and the ranking of the pavements evaluated. Such data sets have not been tested.

Summary. The prediction of pavement roughness is extremely complicated since it is dependent on a large number of factors, which are very difficult to model. For example, the influence of swell in subgrades, consolidation in fills, and faulting in rigid pavement are all factors that are somewhat unpredictable in their time or traffic related impact on roughness. In this study it is assumed that roughness and distress data due to traffic and environmental factors will be available to the engineer from other sources since this investigation is not specifically concerned with predicting distress or serviceability/performance, but is concerned with relating or accommodating distress and performance for use in pavement management systems. The study investigates the availability of such data.

In summary, a number of methodologies have been evaluated to determine their applicability for combining or relating distress to performance. Each of the most promising techniques are discussed in subsequent sections of the report.

OBJECTIVES

The objectives of this research and this report are to develop a meaningful relationship between the occurrence of pavement distress and serviceability and pavement performance, and to develop a model for those relationships that can be used in a predictive pavement design and management system.

SCOPE

This study is involved with the quantification of a most difficult relationship - that of distress to serviceability and performance. It includes an understanding of pavement distress, and identification of the individual and combined effects of the distress on the primary response of the pavement, and effects of various maintenance and rehabilitation strategies on the distress and performance. Pavement management needs distress and performance information in the relationship between the two in order to be effective.

RESEARCH APPROACH

The general research approach used in this project is described briefly in each of the following categories of activities that were undertaken while conducting this study.

Advisory Panel Review

In order to attack this difficult problem we established a project advisory panel to review the history of pavement distress and condition observations and to examine the feasible approaches to this problem. The panel was made up of Dr. Ralph Haas, Dr. Virgil Anderson, Professor Carl Monismith, Dr. Ron Terrel, Mr. Roger V. LeClerc (Washington), Mr. Dale E. Peterson (Utah), Mr. Travis W. Smith (California), Mr. Erland O. Lukanen (Minnesota), Mr. Gerald B. Peck (Texas), and Mr. Robert J. Weaver (New York).

The panel met in Berkeley, California, May 7-9, 1978 and in Austin, Texas, August 6-8, 1978, to discuss the range of possible attacks on performance problems and set the direction for subsequent project work. The input of the panel members individually and as a group was invaluable as was their help in identifying available data for the study. The group, however, cannot be blamed for any shortcomings of this study, which are solely the responsibility of the authors.

Catalogue Pavement Distress

Hudson and McCullough (Ref. 9)[1] have previously developed information concerning categories of pavement distress that have proven useful. Monismith (Ref. 10)[2] has summarized the various distress categories and respective causes for both asphalt concrete and portland cement concrete pavements, and Lytton (Ref. 11)[3] has provided listings of various distress indicators for both pavement types including continuously reinforced pavements and rigid pavements. Haas (Ref. 12)[4] has comprehensively summarized the various manifestations in terms of roughness, distress, and skid resistance. Carter (Ref. 13)[5] has summarized distress manifes-

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[1]Hudson, W.R. and B.F. Mccullough, "Flexible Pavement Design and Management Systems Formulations", NCHRP Report 139, 1973.

[2]Monismith, C.L., "An Overview of Airfield Pavement Design", a paper prepared for presentation at the ASCE Air Transport Speciality Conference, 1977.

[3]Lytton, R.L. and J.P. Mahoney, "Condition Surveys for Pavement Structural Evaluation", paper prepared for presentation at The Transportation Research Board Meeting, 1976.

[4]Haas, R., "Surface Evaluation of Pavements: State-of-the-Art", Proceedings, San Francisco Workshop on Pavement Rehabilitation, Report No. DOT-OS-4-0022, 1974.

[5]Carter, W.H., "Distress Manifestations in Continuously Reinforced Concrete Pavements", Subcommittee Report to Highway Research Board Committee A2B01, Rigid Pavement Design, 1973.

tations in CRCP pavements and has described the contributing causes for each. Using these background references together with the experience of the project staff, a catalogue of pavement distress manifestations was prepared and is presented in Chapter 2.

Define Typical Distress and Serviceability Performance Development Patterns

Figure 3 illustrates possible distress and serviceability performance patterns which are feasible for the various types of distress and for serviceability history. While the representations in this figure are illustrative only, they are compatible with the general experience of the project staff and are consistent with data reported in the literature (Refs. 3, 14[1], 15[2], 16[3], 17[4]).

It is clear from these illustrations that the relationship of distress and performance considering three phases (primary, secondary and time dependent) is very complicated. It is expected that any such relationship will not only involve the individual independent variables (first order) but also interactions between the various independent variables. Various forms of these relationships have been developed and are presented and discussed in subsequent chapters.

Evaluation and Implementation of Methodologies

The various methodologies, discussed previously, were evaluated separately in a three step process as follows:

(1) Development and testing by the project staff using AASHO Road Test information or as necessary by simulation using assumed generated by the project staff.

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[1]Smith, R.P. and B.F. McCullough, "The Use of Condition Surveys, Profile Studies, and Maintenance Studies in Relating Pavement Distress to Pavement Performance", Research Report 12319, Center for Highway Research, The University of Texas at Austin, 1974.

[2]Anderson, D.I., J.C.McBride, and D.E. Peterson, "Field Verification of the VESYS IIM Structural Subsystem in Utah", Proceedings, Fourth International Conference for the Structural Design of Asphalt Pavements, 1977.

[3]Sharma, M.G., W.J. Kenis, T.D. Larson, and W.L. Gramling, "Evaluation of Flexible Pavement Design Methodology Based Upon Field Observations at PSU Test Track", Ibid.

[4]Sharma, J., L.L. Smith, and B.E. Ruth, "Implementation and Verification of Flexible Pavement Design Methodology", Ibid.

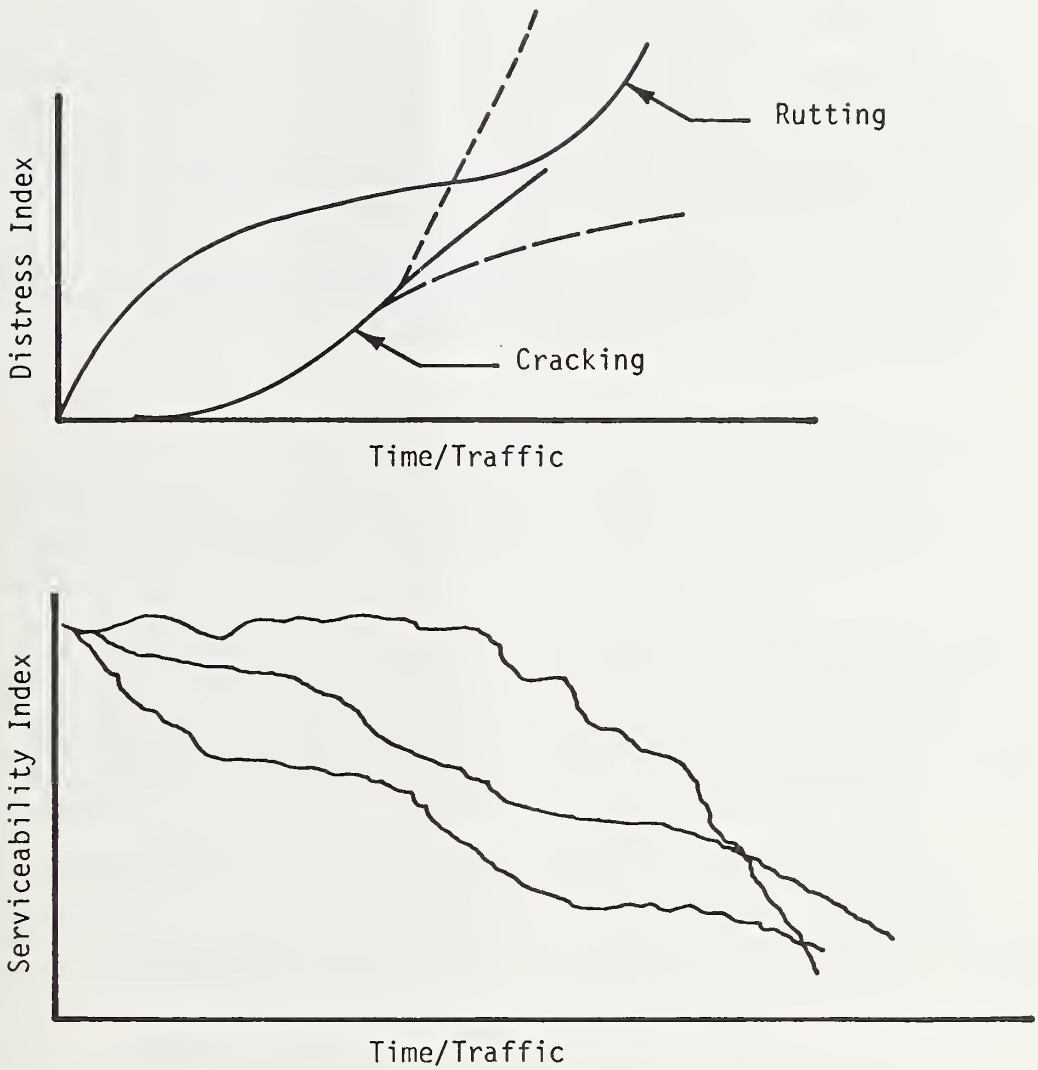

Figure 3 Potential distress and serviceability patterns

(2) Interaction with the Advisory Group by presenting results from step 1 above and obtaining reactions and recommendations

(3) Testing with field data obtained from the states involved in the project.

Based on these evaluations, those models best satisfying project requirements and their use in pavement management systems are presented and discussed in subsequent chapters of the report.

REPORT ORGANIZATION

Chapter 2 on pavement distress and performance measurements includes: definitions of each type of distress investigated in the project, a priority listing of the most important of these distresses, and a discussion of the various measures of serviceability and performance.

The data acquisition phase of the project is described in Chapter 3. Information from special research studies such as the AASHO Road Test, Brampton Test Road, and Texas Transportation Institute Data Base as well as information from the States of Washington, Utah, Minnesota, Illinois, and Georgia are included. In addition, published reports on a number of other special roadway sections are considered in Chapter 3.

Chapter 4 contains a general discussion of the relationships between distress and performance. A description of the typical patterns of distress development and their effect on performance is included in Chapter 4 as well as a discussion of the primary, secondary, and time dependent relationships.

The data analysis and modeling efforts to relate distress to serviceability are included in Chapter 5. A detailed discussion of a series of regression models is included for both time-dependent and time-independent distress/serviceability relationships.

Chapter 6 contains a discussion of the findings of the report, general discussions of adequacy, and suggestions relative to acquisition and analysis of data. Chapter 6 also includes a discussion of overall pavement performance measures as well as the use of performance models in pavement management systems.

Chapter 7 contains recommendations of the project, discussion of significant problems encountered, and a series of problems statements. The problems statements are directed toward the development of more reliable relationships between distress and performance as needed for predicting performance for use in pavement management systems.

CHAPTER 2. PAVEMENT DISTRESS AND PERFORMANCE MEASURES

As a first step toward the development of distress/performance relationships, quantified measures of pavement distress and performance were selected. This process involved several steps, namely (1) definition of terms, (2) cataloging pavement distress manifestations by type, severity, and cause, and (3) selection of variables for study in this project. The results of these activities are reported in this chapter.

DEFINITION OF TERMS

The following definitions of terms relating to pavement distress and performance were selected as appropriate for this study. These definitions follow those used in several previous investigations (Refs. 18[1], 19[2], 20). Only key definitions are reproduced here; the complete set may be found in Appendix A.

1. Distress is a condition of a pavement structure which reduces serviceability or leads to a reduction of serviceability.

2. Distress Manifestations are the visible consequences of various distress mechanisms, which usually lead to a reduction in serviceability.

3. A Distress Mechanism is the physical or chemical process involved in or responsible for distress in pavements.

4. Response is the reaction of a pavement structure to load and environment.

5. Primary Responses are those responses which, when carried past some limiting value, initiate distress.

6. A Response Mechanism is the physical or chemical process responsible for the response of a pavement structure.

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[1]Rauhut, J.B., F.L. Roberts, and T.W. Kennedy, "Models and Significant Materials Properties for Predicting Distress in Zero-Maintenance Pavements", Report No. FHWA-RD-78-84, Interim Report, June 1978.

[2]Hudson, W.R., F.N. Finn, B.F. McCullough, K. Nair, and B.A. Vallerga, "Systems Approach to Pavement Design, System Formulation, Performance Definition, and Material Characterization", Interim Report NCHRP 1-1, submitted by Materials Research and Development, Inc. to NCHRP, HRB, March 1968.

7. Serviceability is the ability of a specific section of pavement to serve traffic in its existing condition.

8. Serviceability Index is a numerical estimate of serviceability based on pavement roughness.

9. Serviceability History is a representation of the serviceability of a pavement over a period of time. Serviceability history is sometimes termed "performance", and is generally given by serviceability index as a function of time or accumulated load.

10. Performance is a measure of accumulated service provided by a facility; i.e., the degree to which a pavement fulfills its purpose.

Pavement performance has in the past generally been directly related to pavement serviceability. That is, performance has been equated with "the area under the serviceability history curve", or "shape of the serviceability curve". Since serviceability is almost universally measured with a serviceability index based on riding comfort or roughness, this usage makes pavement performance a function of pavement roughness. However, many other factors, such as skid resistance, structural adequacy, cracking, etc., may be important in determining the overall adequacy of a pavement. The word "performance" is a natural candidate to describe this overall adequacy. Hence, one of the recommendations of this research effort is that the term "performance" be reserved to reflect the overall adequacy of a pavement (see Chapter 6).

PAVEMENT DISTRESS

In this project, distress is defined as the condition of a pavement structure which reduces serviceability or leads to reduction of serviceability. Occurrence of distress may also lead to maintenance in order to restore serviceability. Distresses that do not directly result in significant losses of serviceability or do not lead to other distresses are not of interest in this study.

Comprehensive Distress Listings

In order to insure that all relevant distress types were considered in this project, an extensive review of existing literature on pavement distress was conducted. Based on this review, four primary sources were chosen to provide a complete catalog of pavement distress (Refs. 18, 21[1],

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[1]Barenberg, E.J., C.L. Bartholomew and M. Herrin, "Pavement Distress Identification and Repair", Technical Report P-6, Construction Engineering Research Laboratory, Department of the Army, March 1973.

22[1], 23[2]). A detailed listing and a discussion of these approaches are presented in Appendix B.

Selection of Distress Types for Further Study

There are many different kinds of distress as summarized in Appendix B. The problem is further complicated because one distress may cause or contribute to the development of another. In addition, it would be time consuming to attempt to model the effects of dozens of distress modes on serviceability/performance. Consequently, the project staff selected for further study those distress types expected to have the most significant influence on pavement performance.

The priority ranking shown in Table 1 (Ref. 18) and discussed in more detail in Appendix B, was expected to be valid for this project with only minor modification. However, as an independent check, Reference 14 was used to make an evaluation of the relative impact of various distress types. The combination of deduct values and utilities for each distress allows an assessment of both the immediate and expected future effect on serviceability/performance. This combination was accomplished as described in Appendix B.

The resulting severity ranking, from most to least severe, and using $\alpha_1=\alpha_2=0.75$, is:

Flexible Pavement Distress:

(1) Failures/mile.
(2) Alligator cracking.
(3) All other cracking.
(4) Flushing Ravelling, Corrugation (very nearly equal).
(5) Rutting.
(6) Patching.

Rigid Pavement Distress:

(1) Pumping.
(2) Surface Deterioration.
(3) Spalling.

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[1]Darter, M.I. and E.J. Barenberg, "Zero-Maintenance Pavement Requirements and Capabilities of Conventional Pavement Systems", Interim Report No. FHWA-RD-76-105, April 1976.

[2]Ledbetter, W.B., R.L. Lytton, et al., "Techniques for Rehabilitating Pavements Without Overlay - A Systems Analysis", Volume 1, Final Report No. FHWA-RD-77-132, September 1977.

TABLE 1 PRIORITY RANKING OF SIGNIFICANT DISTRESSES SELECTED FOR FURTHER STUDY (Reference 23)

Priority Ranking	Flexible	Rigid Pavements: JCP	Rigid Pavements: JRCP	Rigid Pavements: CRCP	Composite Pavements
1	Fatigue Cracking	Fatigue Cracking	Low-Temp. and Shrinkage Cracking	Crack Spalling	Reflection Cracking
2	Rutting	Joint Faulting	Fatigue Cracking	Fatigue Cracking	Fatigue Cracking
3	Low-Temp. Cracking	D-Cracking	Crack Faulting	Low-Temp. Cracking	Rutting
4	Reduced Skid Resistance	Joint Spalling	Joint Spalling	Shrinkage Cracking	Reduced Skid Resistance
5			D-Cracking	Punchouts	
6				Steel Rupture	

(4) Failures/mile.
(5) Cracking.
(6) Faulting.

These rankings are similar to those of Reference 30[1] (Table 1), provided that the different categorization of distress types is taken into account.

In developing a list of important distress types for this study the distress rankings of Reference 30 were given primary consideration, because:

(1) The emphasis on distress categorization by cause rather than appearance is desirable in a theoretically-oriented study.

(2) The inference space is similar to that of interest here (basically primary highways).

(3) It was considered desirable to treat flexible, composite, JCP, JRCP, and CRCP separately, as done in Reference 30.

The project advisory panel made several suggestions about the relative importance of distress types. It was generally agreed that cracking is a serious problem in asphalt pavements, with alligator cracking playing the largest role. The runner-up in this pavement type was permanent distortion in the wheel path caused by studded tires and/or rutting. Flushing and ravelling were also mentioned as important. The largest problem in asphalt pavements was viewed as improper care of joints, leading to faulting and spalling, the latter being considered less important. Wheel path wear and the resulting loss of skid resistance were also considered important, and once again studded tires were believed to contribute heavily to this problem. These suggestions agree in general with the listing in Table 1.

Consequently, the ratings in Table 1 were adopted for this project with the following modifications:

(1) Faulting was given higher priority in JCP, and wheel path wear (reduced skid resistance) was included in all rigid pavements, in accordance with the recommendations of the advisory group.

(2) Several changes were made in order to reconcile these rankings

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[1]Rauhut, J.B. and W.R. Hudson, "Concepts for Optimization of Design for Flexible Pavement Procedures", Report to the Department of Civil Engineering, University of Utah, Austin Research Engineers Inc, November 1974.

with the ratings derived from Reference 29[1] in the previous section. First, failures/mile (potholes) in flexible pavements were considered a composite distress, and treated as fatigue cracking combined with load and water action rather than as a separate category. Failures/mile in rigid pavements were classified as punchouts. Patching and pumping were not considered as distress types, but it is recognized that they may have a significant effect on performance/serviceability, and hence, they were considered for inclusion in the modeling effort in this project. Reduced skid resistance was divided into the categories of wheel path wear and bleeding (flushing).

The resulting priority ranking of distress types for further consideration is recorded in Table 2.

PAVEMENT SERVICEABILITY AND PERFORMANCE

Since the introduction of the pavement serviceability - performance concept at the AASHO Road Test (Refs. 1, 3), a variety of serviceability and performance measures have been developed. Several alternative measures are discussed below.

Pavement Serviceability

Two different methods for evaluating pavement serviceability were involved at the AASHO Road Test (Ref. 3). The Present Serviceability Rating (PSR) is a subjective assessment of the serviceability of the pavement based on the opinions of a panel of raters who ride over the pavement in a standard automobile. Each person rates the pavement on a 0 to 5 scale, with 5 representing a perfect score and 0 an impassable pavement. The Present Serviceability Index (PSI) is an objective estimate of the pavement based on physical measurements of roughness, distress, or other observable characteristics of the pavement. The PSR is thus a subjective measure of pavement serviceability, while the PSI provides an estimate of the serviceability based on correlations of physical measurements to the PSR.

The PSI equations developed at the Road Test by least squares correlation to panel ratings are, for flexible pavement:

$$PSI = 5.03 - 1.91 \log (1+\overline{SV}) - 1.38\, \overline{RD}^2 - 0.01\sqrt{C + P} \qquad (1)$$

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[1]Rauhut, J.B. and P.R. Jordahl, "Effects on Flexible Highways of Increased Legal Vehicle Weights Using VESYS IIM", Final Report No. FHWA-RD-77-134, January 1978.

TABLE 2 PRIORITY RANKING OF SIGNIFICANT DISTRESSES SELECTED

Priority Ranking	Flexible	Rigid Pavements			Composite Pavements
		JCP	JRCP	CRCP	
1	Fatigue Cracking	Joint Faulting	Low-Temp. and Shrinkage Cracking	Crack Spalling	Reflection Cracking
2	Rutting	Fatigue Cracking	Fatigue Cracking	Fatigue Cracking	Fatigue Cracking
3	Low-Temp. Cracking	D-Cracking	Crack Faulting	Low-Temp. Cracking	Rutting
4	Wheel Path Wear	Joint Spalling	Joint Spalling	Shrinkage Cracking	Wheel Path Wear
5	Bleeding	Wheel Path Wear	D-Cracking	Punchouts	Bleeding
6	—	—	Wheel Path Wear	Wheel Path Wear	—

and for rigid pavements:

$$PSI = 5.41 - 1.78 \log (1+\overline{SV}) - 0.09\sqrt{C + P} \qquad (2)$$

Where:

$\overline{SV}$ = mean slope variance from the AASHO Road Test longitudinal profilometer

$\overline{RD}$ = mean rut depth

C+P= amount of major cracking and patching

Strictly speaking, these equations are applicable only to the conditions encountered at the Road Test, since the coefficients were derived through regression.

Most states currently measure pavement serviceability based on roughness measurements; that is, they use some form of Present Serviceability Index. For reasons of efficiency and economy, most states calculate PSI on the basis of roughness measurements. These roughness measurements are typically carried out with an accelerometer device, such as the Mays Meter, which can be mounted in a standard automobile. Some states report these measurements only in terms of counts per mile, or some other direct measure of roughness; others use a serviceability index reported on the standard 0 to 5 scale. The equations relating such serviceability indices to roughness may be based on a variety of analyses. Some states use the AASHO Road Test equations, often with coefficients modified in an attempt to represent local conditions or concern, while others have conducted their own panel ratings and carried out independent correlation analyses. Some states correlate roughness measurements to more sophisticated measurements of road profile, for example, those obtained from the surface dynamics profilometer, which are in turn related to previously established serviceability equations or to the results of panel ratings. Thus, there is no universally accepted standard measure of pavement serviceability. This is discussed further in Chapter 7.

It was not found necessary to prioritize pavement serviceability measures for use in this project. The major reason for this is that each State or agency generally uses only one method (or in the exceptional case, two methods) for quantifying serviceability. In fact, in some cases, <u>no</u> measure of serviceability is recorded. Thus, any reported serviceability variable was considered a potential candidate for modeling.

Pavement Performance

Pavement performance has in the past generally been defined as a summary of accumulation of pavement serviceability index based on objec-

tive measurements of roughness and/or pavement distress. This usage of the work "performance" stems from the work of Carey and Irick (Ref. 1), although their original definition left considerable room for greater generality. There has been no universal agreement on the definition of pavement performance, though. For example, in the recent literature, pavement performance is defined variously as: (1) the ability of a pavement to provide an acceptable level of serviceability with a specified degree of reliability at an assumed level of maintenance (Ref. 24)[1]; (2) allowable repetitions of loading prior to the functional failure of the pavement (Ref. 25)[2]; and (3) the probability that a critical life of the pavement will be achieved based on the trend of deflection changes up to the onset of critical conditions (Ref. 26)[3].

At the beginning of this project, the research staff felt that performance should be quantified as accumulated serviceability. That is, the serviceability history of a pavement section with time or load was taken as the measure of performance for that section. Thus, the major modeling effort for this project was directed toward performance as the record of serviceability over time. Certain difficulties were encountered with this approach during the conduct of the research, as described in subsequent chapters. The project staff concluded that it would be desirable to use a more comprehensive measure of pavement performance; however, no generally accepted measure could be found. Hence, part of the research effort was directed toward establishing more comprehensive performance measures, as discussed in Chapter 6.

EVALUATION OF EXISTING MECHANISTIC DISTRESS MODELS

Existing models for pavement structural analysis are either design or research oriented. In general, the more input variables considered in describing the material properties or the responses of a pavement structure, the more complex the analytical procedure. The excessive computation time required for the analysis as well as the laboratory equipment required for the testing of the material properties some-

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[1]Kenis, W.J., "Predictive Design Procedures, VESYS User's Manual", Report No. FHWA-RD-77-132, September 1977.

[2]Crawford, J.E. and M.G. Katona, "State-of-the-Art for Prediction of Pavement Response", Report No. FAA-RD-75-183, U.S. Army Engineer Waterways Experiment Station, September 1975.

[3]Kennedy, C.K. and N.W. Lister, "Prediction of Pavement Performance and the Design of Overlays", Transport and Road Research Laboratory Report 833, Crowthorne, Berkshire, Great Britain, 1978.

times makes the research oriented type of analytical tool impractical for everyday design purposes. However, the research oriented models can often eliminate some over-simplified assumptions and can be more realistic in modeling the true behavior of the materials as well as the structural response of the pavement. Since the goal is to be able to relate predicted distress value to pavement performance, the models selected should be able to predict as accurately as possible the occurrence of the major distresses for each pavement type. The basic criteria used to select the candidate models are: (1) the model can realistically simulate the material properties as well as the structural response of the pavement, (2) the model should be able to predict specifically the occurrence and the severity of the major distresses that are common to that pavement type. The evaluation of the candidate models was conducted separately by types of pavement, which include flexible pavement, jointed concrete pavement (JCP), jointed reinforced concrete pavement (JRCP), and continuously reinforced concrete pavement (CRCP). Composite pavements were analyzed in conjunction with flexible pavements.

The evaluation of existing distress models is discussed in Appendix C. Models selected for each pavement type are listed in Table 3.

TABLE 3 SUMMARY OF MODELS

	Pavement Type	Selected Model	Other Models Considered
(1)	Flexible Pavement and Composite Pavement	VESYS A	VESYS IIM, PDMAP, COLD
(2)	Jointed Concrete Pavement	JCP-1	
(3)	Continuously Reinforced Concrete Pavement	CRCP-2 & ELSYM5	LAYER5, LAYER15, LAYIT BISAR, CRCP-1
(4)	Jointed Reinforced Concrete Pavement	JRCP-2	JRCP-1

The models selected have been exercised in the recent past by members of the project staff to evaluate their utility. Results from the solutions of these models enable us to determine the quantity and the magnitude of each distress given a pavement structure design, loading conditions, and environmental factors. These distresses can then be related to serviceability through the AASHO equations, the time-independent models discussed in Chapter 5, or a similar set of equations developed by the individual agency involved.

Either mechanistic or empirical models can be used for predicting

pavement distress. The primary purpose of this investigation, however, is to relate distress to performance. For this purpose, distress need not be predicted, but can be measured in the field. Collection of actual field records of pavement distress and serviceability is discussed in Chapter 3.

CHAPTER 3. DATA COLLECTION

Records of pavement distress, serviceability and performance were obtained from several sources, as discussed below. These data were employed in the modeling effort under Tasks B through E. A brief description of each data source is given in this chapter, along with a sample of the actual data obtained.

It should be pointed out here that an exhaustive effort was made through our advisory panel and through our own extensive experience in this area to find all possible available pavement performance, distress and serviceability data with a significant time and traffic history.

The amount of data available with adequate time and traffic history, and including both distress and performance is extremely limited. In nearly every case some aspect of the needed information was found to be missing as outlined herein in detail. Long term data base observations are needed to solve the problems posed here.

SPECIAL RESEARCH STUDIES

The initial modeling efforts for this project were based on data records from special research studies. Such data were chosen for the preliminary investigations under Task B because they include fairly comprehensive records of a large number of distress types for both flexible and rigid pavements, obtained under controlled conditions, along with the needed serviceability and performance data.

AASHO Road Test

The AASHO Road Test provides the most comprehensive set of pavement distress and performance data yet collected. Hence, these records are natural candidates for distress/performance modeling.

The data recorded at the AASHO Road Test were cataloged and stored in a series of data files, as reported in Reference 3. The data files available for use in this project included 4199-F, 4292-R, and 7322-D. These records involve data for both flexible and rigid tests sections, and include plots of serviceability and distress variables as measured biweekly (both microfilm and paper copy versions), as well as tabular numerical records of distress and serviceability data.

Samples of these data records are provided in Figures 4 and 5 and Table 4. The pavement distress, performance, and time variables recorded in these data sets are summarized in Tables 5 and 6 for flexible and rigid pavements, respectively.

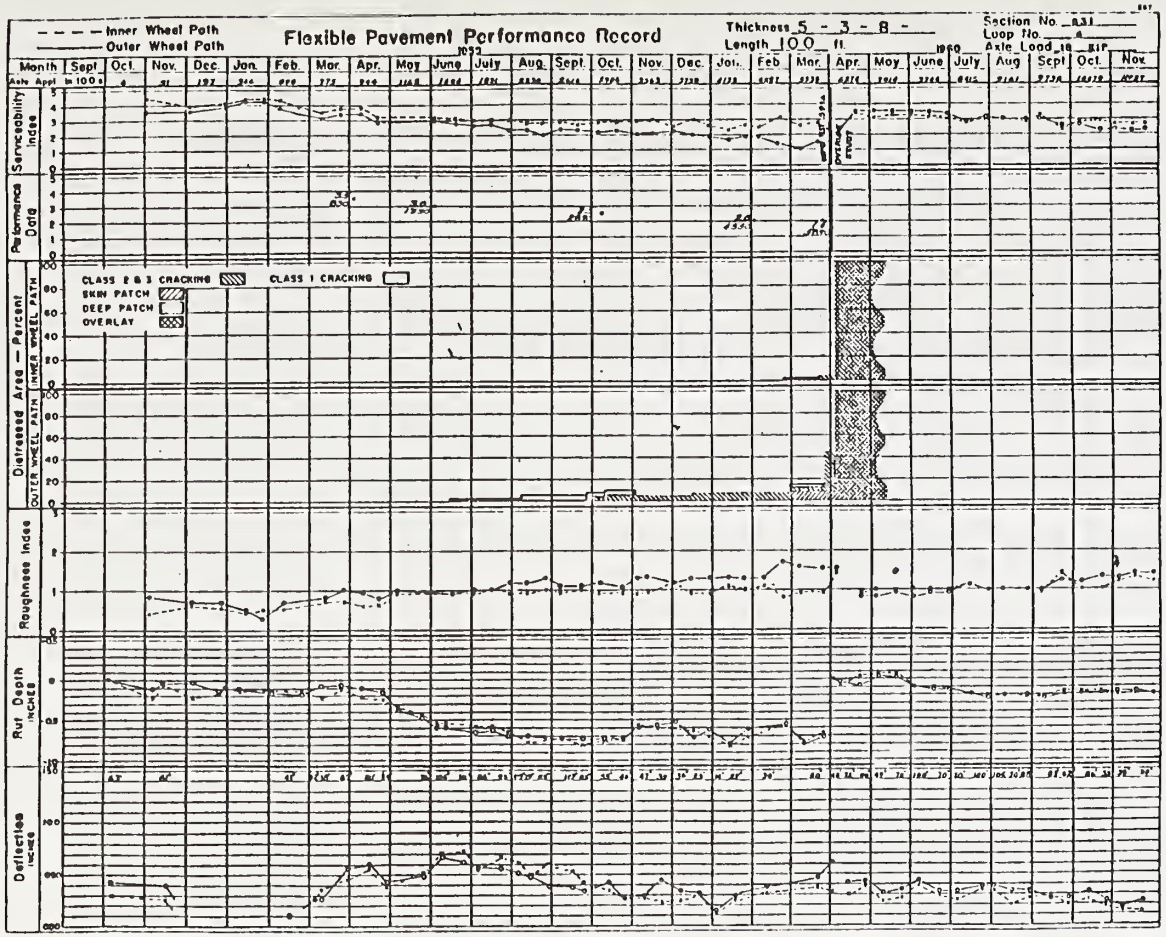

Figure 4 Example flexible pavement performance record AASHO Road Test

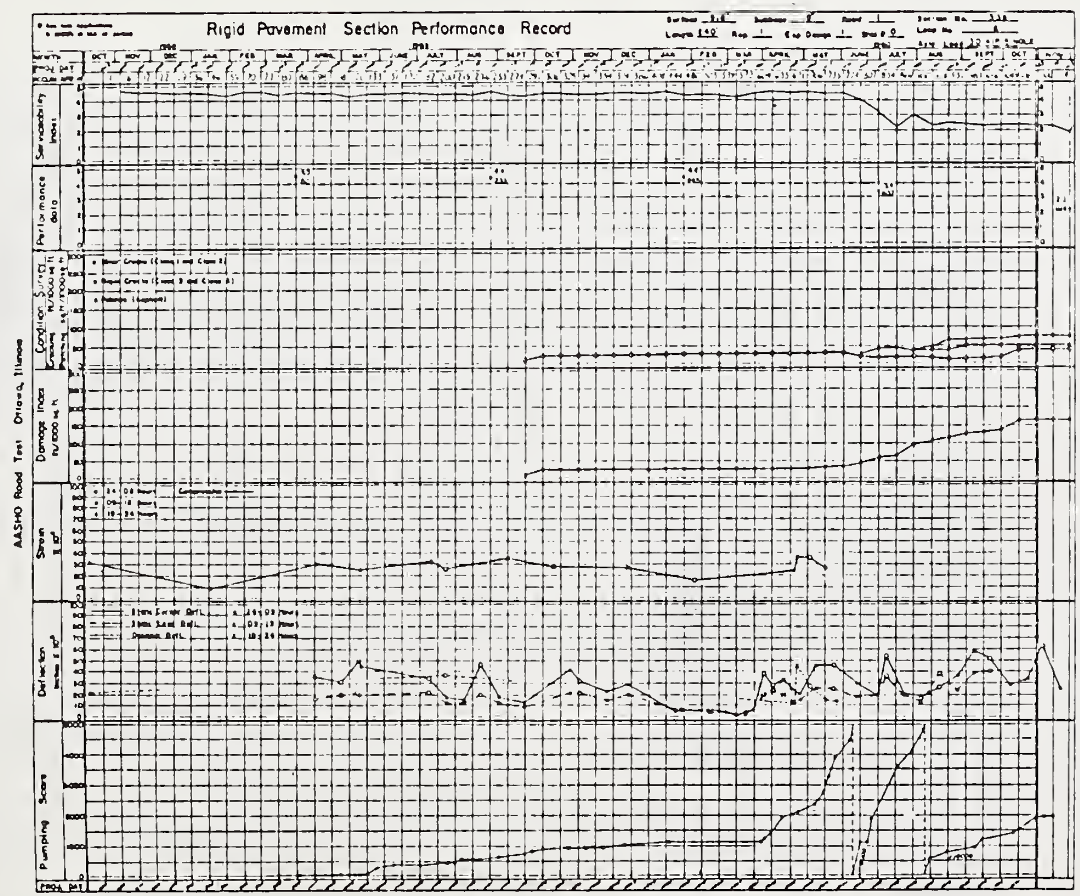

Figure 5 Example rigid pavement performance record, AASHO Road Test

TABLE 4 EXAMPLE AASHO ROAD TEST DATA FILE

7322	612	1	322					13	35			105	110	43	40	41		121	100	857
7322	612	1	322							81	81	99	98			40			3000	871
7322	612	1	322					81	37			110	100	41	37	39		121	5550	885
7322	612	1	322							108	122	107	113			39			10806	899
7322	612	1	322									104	108			39			14362	913
7322	612	1	322					16	45	102	81	101	110	42	36	39		121	17767	927
7322	612	1	322					28	45			101	110	41	36	38		121	23801	941
7322	612	1	322					21	29	102	81	98	113	41	39	40		121	29862	955
7322	612	1	322					27	60			95	111	39	34	36		121	34755	969
7322	612	1	322					29	34	108	108	100	117	39	38	38		121	38843	983
7322	612	1	322					49	54			105	120	39	34	36		121	39967	997
7322	612	1	322					37	68	81	81	110	127	37	32	35		121	42911	1011
7322	612	1	322	10				32	89			127	147	37	20	32		121	48955	1025
7322	612	1	322	10				98	110	108	183	135	157	29	25	27		121	53970	1039
7322	612	1	322		80					81	102	143	166			24			60026	1053
7322	612	1	322	197	500					138	150	150	177			21			66756	1067
7322	612	1	322	243	500			91	183			160	183	24	14	19		121	76456	1081
7322	612	1	322	237	400	40	40	113	248	308	291	150	180	25	14	19		121	84920	1095
7322	612	1	322	157	312	40	40					160	193		16	20			90794	1109
7322	612	1	322	157	312	40	40					160	183		18	21			97794	1123
7322	612	1	322	157	312	40	40					160	190		20	22			106233	1137
7322	612	1	322	157	312	40	40	174	190	204	241	135	145	24	22	23		121	114637	1151
7322	612	1	322					12	115	98	92	110	100	44	29	34	•	•121	122540	1165
7322	612	1	322					66	45			107	100	33	36	35	•	•121	133135	1179
7322	612	1	322					60	48	100	114	110	100	34	36	35	•	•121	142067	1193
7322	612	1	322					70	58			110	100	33	35	34	•	•121	152701	1207
7322	612	1	322					60	64			107	100	34	33	34	•	•121	162360	1221
7322	612	1	322					66	39	82	87	103	100	33	37	35	•	•121	170017	1235
7322	612	1	322					87	43			107	100	34	36	35	•	•121	175740	1249
7322	612	1	322					73	52			113	100	33	35	34	•	•121	186731	1263
7322	612	1	322					74	52			103	100	33	35	34	•	•121	193670	1277
7322	612	1	322					63	38	98	90	110	105	34	37	35	•	•121	203313	1291
7322	612	1	322					64	46			117	110	34	36	35	•	•121	218597	1305
7322	612	1	322					69	46			117	110	33	36	34	•	•121	236154	1319
7322	612	1	322					69	59			110	110	33	34	34	•	•121	240772	1333
7322	612	1	322					68	54			120	113	33	35	34	•	•121	250511	1347
7322	612	1	322					73	72	117	155	110	107	33	33	33	•	•121	260351	1361
7322	612	1	322					68	59			113	103	33	34	34	•	•121	295407	1375
7322	612	1	322					69	54			110	110	33	35	34	•	•121	311374	1389
7322	612	1	322					71	47			107	103	33	36	34	•	•121	320870	1403
7322	612	1	322					57	46	91	96	120	110	33	36	35	•	•121	346935	1417
7322	612	1	322					101	78			113	110	30	33	31	•	•121	361157	1431
7322	612	1	322					68	47			117	113	34	36	35	•	•121	375183	1445
7322	612	1	322					83	57	96	93	123	117	31	35	32	•	•121	395241	1459
7322	612	1	322					90	70			120	117	30	32	32	•	•121	406759	1473
7322	612	1	322					66	57			117	113	34	35	34	•	•121	433643	1487
7322	612	1	322					85	77			123	120	31	31	31	•	•121	439589	1501
7322	612	1	322					65	60	90	111	113	113	34	34	34	•	•121	453266	1515
7322	612	1	322					83	57			127	120	31	34	32	•	•121	467762	1529
7322	612	1	322					64	59			127	120	33	33	33	•	•121	483067	1543
7322	612	1	322					61	47	109	101	123	117	33	36	34	•	•121	490392	1557
7322	612	1	322					87	75			127	120	30	32	31	•	•121	514596	1571
7322	612	1	322					44	63	171	87	117	117	36	35	35	•	•121	529140	1585
7322	612	1	322					81	45			113	113	38	36	34	•	•121	540674	1599
7322	612	1	322					58	55	120	88	117	113	34	35	34	•	•121	552759	1613
7322	612	1	322					69	79	100	96	117	113	33	32	33	•	•121	558892	1614

TABLE 5 FLEXIBLE PAVEMENT VARIABLES, AASHO ROAD TEST

Distress and Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Cracking, Classes 1,2,3	PSI	AASHO Road Test
(sq.ft.per 1000 sq.ft)	(dimensionless)	Index Day
Rut Depth	"Performance Data"	
(inches)	(=simultaneous PSI	Month
Slope Variance	and accumulated axle	Accumulated
(dimensionless)	load values).	Axle Load Applica-
Roughness Index		cations
(dimensionless)		(dimensionless)
Patching		
(sq.ft.per 1000 sq.ft.)		
Deflection		
(mils)		

TABLE 6 RIGID PAVEMENT VARIABLES, AASHO ROAD TEST

Distress and Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Cracking, Classes 1,2,3 and 4 and Cracking Index (ft.per 1000 sq.ft.)	PSI (dimensionless)	AASHO Road Test Index Day
Pumping Index (cubic in.per in.)	"Performance Data" (=simultaneous PSI and accumulated axle load values)	Month
Slope Variance (dimensionless)		Accumulated Axle Load Applications (dimensionless)
Patching (sq.ft.per 1000 sq.ft.)		
Deflection (mils)		
Surface Strain (microinches per inch)		

Brampton Test Road

In 1965, thirty-six 200 ft. (61 m.) test sections of flexible pavement were constructed on Highway 10 north of Brampton, Ontario. Periodic measurements of various pavement distress, performance and related variables were made throughout the rest of the decade, and were continued into the late 1970s on several surviving test sections. Dr. W.A. Phang of the Ontario Ministry of Transportation and Communications graciously made available some of these distress and performance records for use in this project.

Examples of the Brampton Test Road data are provided in Figures 6 through 8 and Table 7. Table 8 lists the distress, performance and time variables measured at the Brampton Test Road.

TTI Data Base

For several years Dr. R.L. Lytton and his collegues at the Texas Transportation Institute have been collecting data from the Texas State Department of Highways and Public Transportation for use in a number of research efforts. These data records have been compiled in a computerized data base including pavement condition, traffic and environmental information on hundreds of sections of interstate, primary and secondary roadways throughout Texas. Dr. Lytton generously provided a portion of this data base to ARE Inc for use in this project.

Data on over 100 sections of flexible and composite pavements were provided. Examples of these data records are shown in Figures 9 and 10, and Table 9 lists the distress, performance and time variables included. Note that this is one of the most comprehensive data bases used in this research effort.

STATE TRANSPORTATION AGENCIES

To assure a wide range of applicability for the results of this project, and to achieve a firm basis for their practical use, data records were solicited from several State highway agencies. These data records were collected under Task C for use primarily in that Task, although considerable overlap occurred with Tasks B through E. The states contacted were those known to have records available from special research studies or from pavement management system development and implementation.

Washington

Washington has been involved in the development and implementation

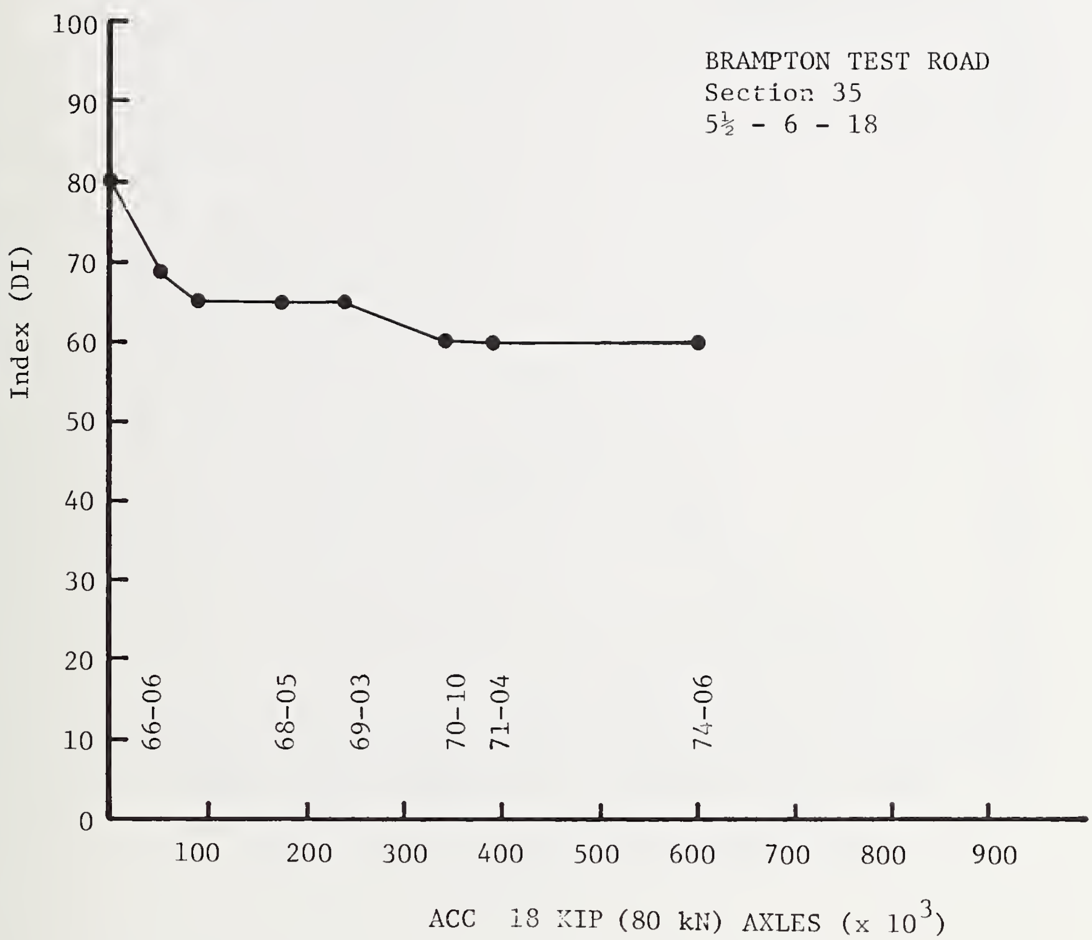

Figure 6 Example distress index (DI) plot, Brampton Test Road

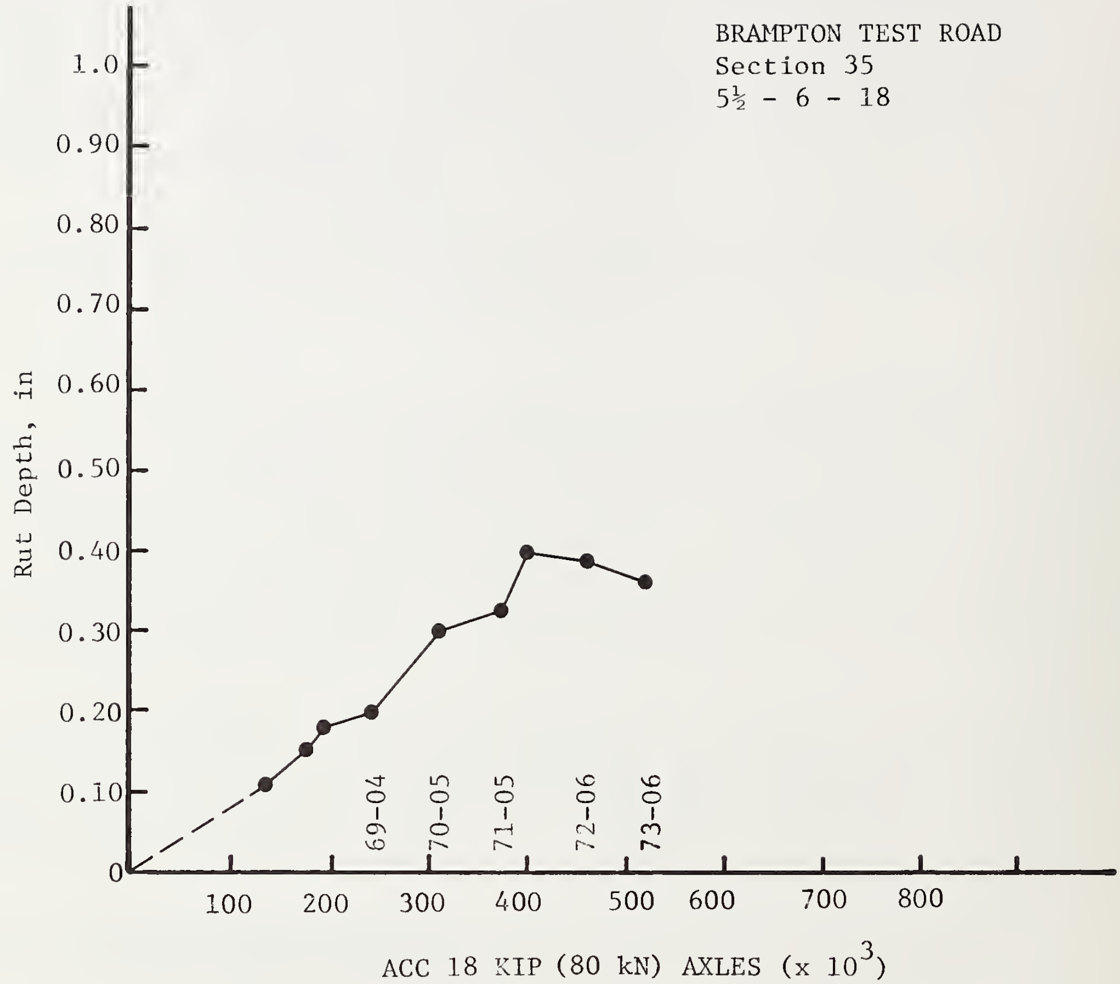

Figure 7 Example rut depth plot, Brampton Test Road

1 in. = 2.54 cm.

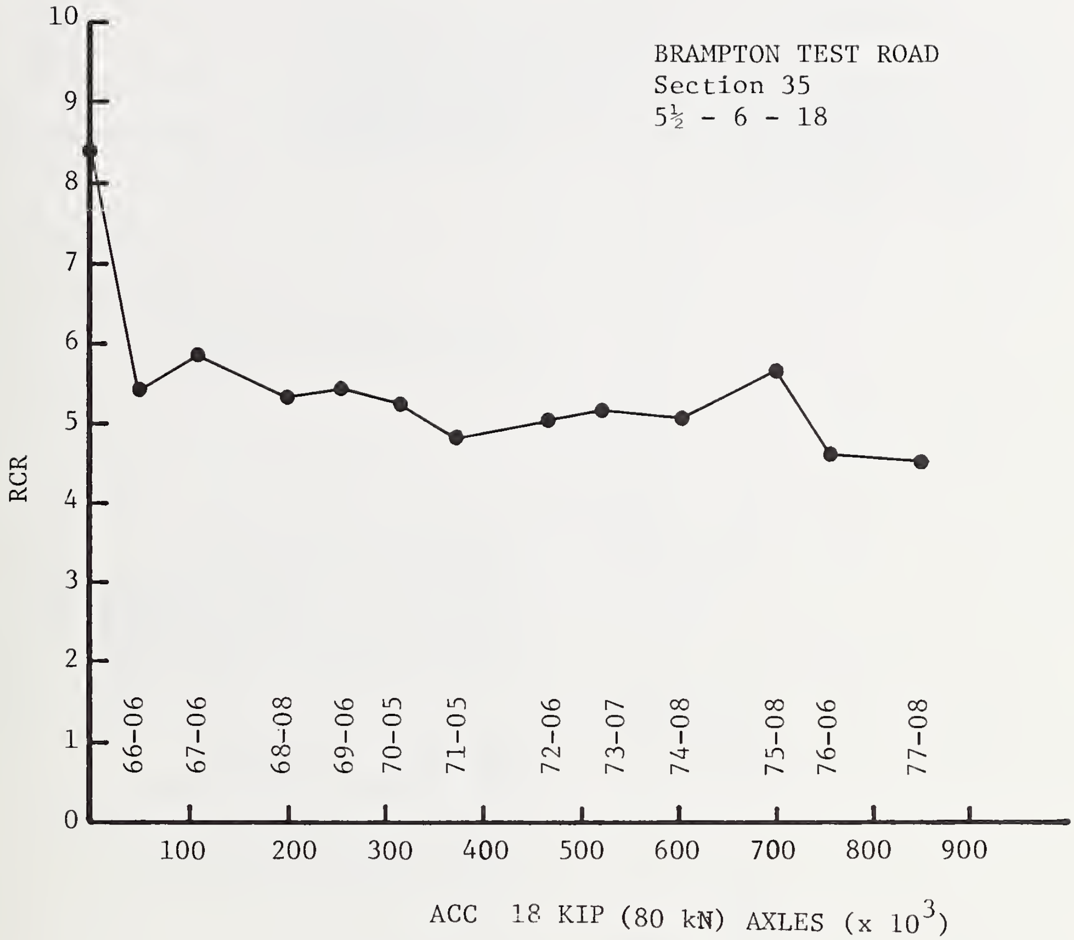

Figure 8 Example riding comfort rating (RCR) plot, Brampton Test Road

TABLE 7 EXAMPLE PAVEMENT CONDITION RECORD, BRAMPTON TEST ROAD

Section 35 5½-6-18

DATE			DISTRESS INDEX		
YR	MO	RCR	DM	DI	DESCRIPTION
65	10	8	80	0.0	
66	06	6	69	0.0	
67	03	6	66	15.0	Rutting V.SL. throughout Trans half V.SL. FEW
68	05	6	66	15.0	
69	03	6	66	15.0	
70	10	5	60	15.0	Ravelling V.SL. FEW Rutting SL. throughout Longit. W.P. single V.SL. FEW Pave. edge single V.SL. FEW Trans half V.SL. FEW
71	04	5	60	15.5	Ravelling V.SL. FEW Rutting SL. throughout Longit. W.P. single V.SL. freq. Pave. edge single V.SL. inter. Trans partial V.SL. FEW Trans half V.SL. FEW
74	06	5	60	18.3	Ravelling SL. FEW Rutting SL. throughout Longit W.P. single V.SL. extensive Midlane single V.SL. freq. Pave. edge V.SL. extensive Trans half V.SL. FEW

TABLE 8 FLEXIBLE PAVEMENT VARIABLES, BRAMPTON TEST ROAD

Distress and Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Rut Depth (inches)	Riding Comfort Rating (RCR), also known as Pavement Performance Rating (PPR) (dimensionless)	Calendar day
Ravelling (severity and extent)		Accumulated Equivalent 18-kip Axle Loads (dimensionless)
Cracking longitudinal, transverse and edge (severity and extent)		
Deflections (mils)		
Frost Depth (inches)		
Distress Index (DI), a combination of RCR and defect values for various distresses		

LOCATION

SECTION ID NO: 351
DISTRICT NO: 4
COUNTY NO/NAME: 118/HUTCHINSON
CONTROL-SECTION: 557- 2
HIGHWAY: SH 152
MILE POINTS: 6.023 - 7.870
LANE: L

FROM POST 8 TO POST 6

ENVIRONMENTAL - 20 YEAR SUMMARY (1955-1974)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	AVG
THORNTHWAITE INDEX:	-	-	-	-	-	-	-	-	-	-	-	-	-23.4
MEAN TEMPERATURE:	38	42	49	59	66	76	80	78	71	61	48	40	59.1
PRECIPITATION:	0.4	0.8	1.1	1.2	2.9	3.1	3.4	2.3	1.6	1.4	0.7	0.6	19.6
WET F-T CYCLES:	1	2	1	0	0	0	0	0	0	0	1	1	7
TOTAL F-T CYCLES:	19	16	9	2	0	0	0	0	0	0	9	18	73
DIST TEMP CONSTANT:	-	-	-	-	-	-	-	-	-	-	-	-	16.7
SOLAR RADIATION:													

PAVEMENT CONDITION SURVEY

PVMT RATING	77	76	75	74	73
PRS	90	90	100	70	57
RUTT	1 SL	1 SL			
FLUSH	1 SL	1 SL			2 SL
CORR					
RAVEL	1 SL	1 SL			2 SL
ALLG CR					
LONG CR				2 MO	2 MO
TRANS CR				3 MO	2 MO
CRACKS				NS	NS
PATCHING				1 G	
FAIL/MI	0	0	0	0	0
*** ***	*** ***	*** ***	*** ***	***	***
SRS - P					
SRS - U	76	80	67	71	89
RRS	77	82	75	77	65
DRS	87	87	70	70	90
TSRS	82	84	80	80	88
*** ***	*** ***	*** ***	*** ***	***	***
P. U. R.	2	1	1	4	4

TRAFFIC

1975 ADT: 1500
1975 PERCENT TRUCKS: 9.5
9/59-10/78 VEHICLES: 5433653
9/59-10/78 18K AXLES: 349438
EXTRAPOLATED FOR OVER 3 YEARS.

SKID NUMBER

DATE	AVG	LOW	HIGH
7/76	43	31	48
7/76	40	34	46

SERVICEABILITY INDEX

YR	MEAN	STD DEV	N	CV	LOW	HIGH
77	3.99	0.183	9	4.6	3.70	4.30
76	4.36	0.230	9	5.3	4.00	4.60
75	4.06	0.207	10	5.1	3.60	4.30
74	3.86	0.283	8	7.3	3.30	4.10
73	3.67	0.231	10	6.3	3.10	3.90

DYNAFLECT DEFLECTION BASIN

DATE	WI MEAN	WI STD DEV	SCI MEAN	SCI STD DEV
10/29/75	0.909	0.051	0.372	0.032

	PAVEMENT	SUBGRADE
STIFF. COEFF.:	0.601	0.280

STRUCTURAL SECTION

LAYER	DESCRIPTION	JOB	WORK TYPE	DATE	AGG / SOIL TYPE	ITEM CL	TYPE GR	AGG. RATE	ADMIX	PCNT	APPL RATE	THICKNESS CENT	THICKNESS EDGE	TTC	LL	PI
1	S HMAC	12	MAINT	5/75		340			AC	6.3		1.4	1.4			
2	S SC	11	MAINT	9/65		316	B 4	110	OA-175		0.3	0.3	0.3			
3	S ST	8	WID-REC	9/59		305	B 4	120	OA-175		0.3	0.2	0.2			
4	S ST	8	WID-REC	9/59		305	B 4	80	OA-175		0.3	0.4	0.4			
5	B FLEX	8	WID-REC	9/59	SILICEOUS	213A						8.0	8.0			
6	SG CLAY	8	WID-REC	9/59	CL.ML.GM									3.8	26.9	9.1

Figure 9 Example flexible pavement data record, TTI Data Base

LOCATION

SECTION ID NO:	39
DISTRICT NO:	1
COUNTY NO/NAME:	117/HUNT
CONTROL-SECTION:	9-13
HIGHWAY:	IH 30
MILE POINTS:	27.800 - 29.800
LANE:	R

POST 107 TO H-H CO.LINE

ENVIRONMENTAL - 20 YEAR SUMMARY (1955-1974)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	AVG
THORNTHWAITE INDEX:	-	-	-	-	-	-	-	-	-	-	-	-	41.6
MEAN TEMPERATURE:	41	45	53	63	71	78	83	82	76	66	53	45	63.1
PRECIPITATION:	2.2	2.7	3.3	5.1	5.2	3.5	3.0	2.7	5.3	4.1	2.9	3.0	42.6
WET F-T CYCLES:	2	1	1	0	0	0	0	0	0	0	0	1	6
TOTAL F-T CYCLES:	13	9	4	0	0	0	0	0	0	0	3	11	40
DIST TEMP CONSTANT:	-	-	-	-	-	-	-	-	-	-	-	-	22.8
SOLAR RADIATION:													

PAVEMENT CONDITION SURVEY

PVMT RATING	77	76	74
PHS	69	76	76
RUTT	2 SL	2 SL	2 SL
FLUSH	2 MO		
CORR			
RAVEL		1 SL	
ALLG CR			
LONG CR	1 MO	1 SL	1 SL
TRANS CR	1 MO	2 MO	2 MO
CRACKS	NS	NS	NS
PATCHING			1 F
FAIL/MI	0	0	0
SRS - P			
SRS - U	71	57	77
RRS	80	75	85
DRS	83	63	80
TSRS	86	90	82
P. U. R.	4	5	4

TRAFFIC

1975 ADT:	10175
1975 PERCENT TRUCKS:	20.6
9/52-10/78 VEHICLES:	30889952
9/52-10/78 18K AXLES:	4899832

SKID NUMBER

DATE	AVG	LOW	HIGH
3/75	25	22	29
7/74	33	30	36

SERVICEABILITY INDEX

YR	MEAN	STD DEV	N	CV	LOW	HIGH
77	3.30	0.355	10	10.8	2.70	3.80
76	3.45	0.584	10	16.9	2.10	4.00
74	3.44	0.230	9	6.7	2.90	3.60

DYNAFLECT DEFLECTION BASIN

	WI		SCI	
DATE	MEAN	STD DEV	MEAN	STD DEV
8/ 9/76	0.563	0.093	0.031	0.006

	PAVEMENT	SUBGRADE
STIFF. COEFF.:	1.381	0.168

STRUCTURAL SECTION

LAYER	DESCRIPTION		JOB	WORK TYPE	DATE	AGG / SOIL TYPE	ITEM CL	TYPE GR	AGG. RATE	ADMIX	PCNT	APPL RATE	THICKNESS CENT	THICKNESS EDGE	TTC	LL	PI
1	S	HMAC	28	MAINT	8/67	SILICEOUS	340	D		AC	4.9		1.5	1.5			
2	S	HMAC	28	MAINT	8/67	SILICEOUS	340	B		AC	4.7		2.5	2.5			
3	S	PCC	4	NEW CON	9/52		320						10.0	10.0			
4	SB	FLEX	4	NEW CON	9/52		SP	B					6.0	0.0			10.0
5	SG	CLAY	4	NEW CON	9/52	CL.CH									5.6	61.3	39.1

Figure 10 Example composite pavement data record, TTI Data Base

TABLE 9 FLEXIBLE AND COMPOSITE PAVEMENT VARIABLES, TTI DATA BASE

Distress and Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Condition Survey: Rutting, Flushing Corrugations, Raveling, Cracking (Alligator, transverse and longitudinal), Failures per mile. (severity & extent)	PSI (dimensionless)	Year
Patching (severity & extent)	Standard Deviation and Coefficient of Variation of Annual PSI Measurements (dimensionless)	ADT (dimensionless)
Pavement Rating Score (PRS) (dimensionless)		Accumulated Vehicle trips (dimensionless)
Skid Number		Accumulated Equivalent 18-kip Axle Loads (dimensionless)
Deflections, Dynaflect and Surface Curvature		
Index (mils)		

of a pavement management system for several years (Refs. 44[1], 45[2]). As a result, they have compiled an extensive pavement condition and performance data base, a major portion of which has recently been transferred to a computerized master file system. Messrs. Roger LeClerc and Tom Nelson of the Washington Department of Highways were most helpful in providing a printout of the Master File listing for all sections of Interstate 5. In addition, they arranged for the transfer of selected data records to ARE Inc in coded form on magnetic tape to expedite our analysis.

Examples of the Master File listings for flexible and rigid pavement sections are reproduced in Tables 10 and 11. Tables 12 and 13 list the distress, performance and time variables recorded for flexible and rigid pavement sections, respectively.

Utah

Utah has also done considerable work in the area of pavement management during the last decade (Refs. 44, 46[3]), and has thus collected several years of pavement condition and performance data, much of which is stored in a computerized data base. Mr. Dale Peterson of the UTAH DOT[4] was most helpful in arranging for a data sample for use in this project.

The data received from Utah include pavement condition and performance records for fifteen flexible pavement sections. Examples of these data are reproduced in Figures 11 and 12. The pavement distress, performance and time variables included in the UDOT data sample are listed in Table 14.

Minnesota

In the early 1960s, Minnesota began a major performance study of flexible pavements. This study, known as Investigation 183, produced an annual record of pavement condition and performance variables over the period 1964-1977. Mr. Erland Lukamen of the Minnesota DOT graciously

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[1]Terrel, R.L. and R.V. LeClerc, "Pavement Management Workshop, Tumwater, Washington", Report No. FHWA-TS-79-206, September 1978.

[2]Finn, F.N., R. Kulkarni and K. Nair, "Pavement Management System Feasibility Study", Final Report prepared for the Washington Highway Commission by Materials Research and Development, Oakland, California, August 1974.

[3]Anderson, D.I., D.E. Peterson and L.W. Shepherd, "Improvement of Utah's Flexible Pavement Performance System", Report No. UDOT-MR-76-1, Final Report, March 1976.

Table 10 EXAMPLE FLEXIBLE PAVEMENT MASTER FILE LISTING, WASHINGTON DOT

[illegible]	[illegible]	END	D	SR	[illegible]	[illegible]	LENG	FC	[illegible]	HT	[illegible]	RSH	LSH	CONT#	TY	S	TYP	M	YR	BASE	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	6	234	1	5	18774	[illegible]	234	[illegible]	L	34	48	10	10	07441	0	[illegible]	[illegible]	1	56	31	[illegible]0100	[illegible]	[illegible]	[illegible]

[illegible]	SIDE	FCSMP	RUTT WEAR	RAVL SAGS	ALG CRK	RAV FLU	LONG CRK	TRNS CRK	PTCH	BUMPS	RIDE SPEED
73	1	80	1	11	1N	31	1N	11	1N	0	
69	1	180	1	12	1N	31	31	11	1N	0	
69	1	280	1	12	1N	31	10	11	1N	0	
71	1	55	1	11	1N	31	21	31	1N	647	50
71	1	155	1	11	1N	31	21	31	1N	665	50
71	1	255	1	11	1N	31	1N	31	1N	756	50
73	1	55	1	11	1N	31	11	11	1N	326	50
73	1	155	1	11	1N	31	11	11	1N	309	50
73	1	255	1	11	1N	31	11	11	1N	391	50
75	1	100	1	11	1N	31	11	21	11	495	50
75	1	200	N	11	11	31	11	21	11	622	50
75	1	221	N	12	1N	12	11	21	11	643	50
75	1	237	N	12	1N	12	11	21	11	643	50
75	1	300	N	12	1N	12	1N	1N	1N	420	50
77	1	100	1	1N	1N	11	11	22	1N	435	50
77	1	200	1	11	1N	11	11	22	1N	618	50
77	1	237	1	1N	1N	21	11	22	1N	594	50
77	1	300	1	1N	1N	11	1N	1N	1N	426	50

TABLE 11 EXAMPLE RIGID PAVEMENT MASTER FILE LISTING WASHINGTON DOT

CS	BEG	END	D	SR	BEG	END	LENG	FC	L	HT	ROW	RSH	LSH	CONT#	TY	S	THK	H	YR	BASE	76 ADT	GRW	SU	CH
2904	219	878	1	5	23340	23999	659	85	L	32	38	10	0	07674	0	7	75	-	66	10	15500	79	5	12

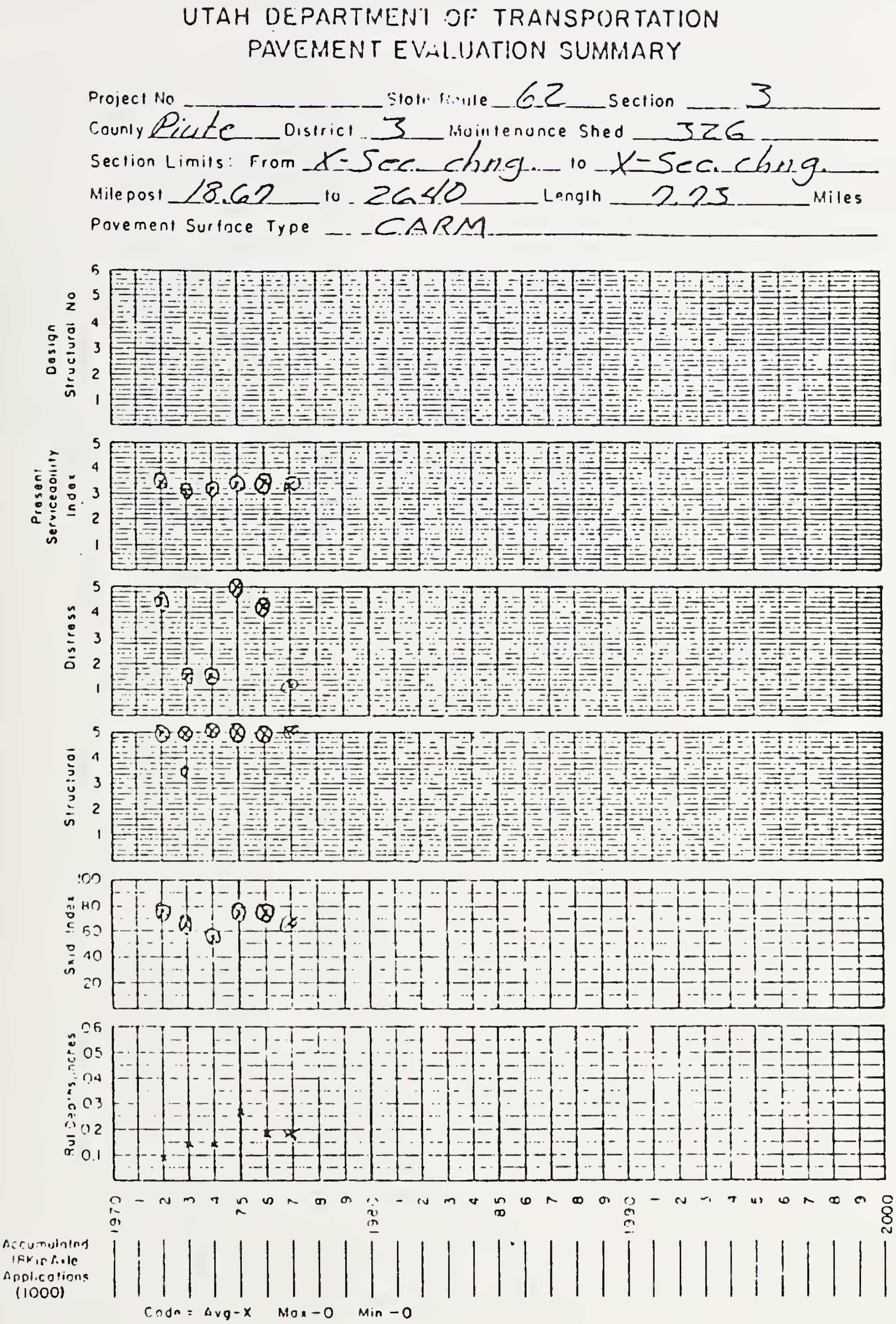
GFN	SIDE	ECSMP	RUTT WEAR	BLOW UPS	CRKS	RVL	JNT SPAL	PUMP BLOW	FALT WARP	PTCH	BUMPS	BUMP SPEED
69	L	300		N	1N	12	1N	1N	1N	1N	0	
69	L	400		N	1N	12	1N	1N	1N	1N	0	
69	L	500		N	11	12	1N	1N	1N	1N	0	
69	L	600		N	1N	12	1N	1N	1N	1N	0	
69	L	700		N	1N	12	1N	1N	1N	1N	0	
69	L	800		N	1N	21	1N	1N	1N	31	0	
69	L	900		N	1N	21	1N	1N	1N	1N	0	
71	L	300	N	N	1N	11	11	1N	11	1N	730	50
71	L	400	N	N	1N	11	11	1N	11	1N	474	50
71	L	500	N	N	1N	11	11	1N	11	1N	427	50
71	L	600	N	N	1N	11	11	1N	11	1N	488	50
71	L	700	N	N	1N	11	11	1N	11	1N	492	50
71	L	800	N	N	1N	11	11	1N	11	21	493	50
71	L	900	N	N	1N	11	11	1N	11	1N	429	50
73	L	300		N	1N	31	1N	1N	11	1N	567	50
73	L	400		N	1N	31	1N	1N	11	1N	276	50
73	L	500		N	1N	31	1N	1N	11	1N	290	50
73	L	600		N	1N	31	1N	1N	11	1N	319	50
73	L	700		N	1N	31	1N	1N	11	31	300	50
73	L	800		N	1N	31	1N	1N	11	1N	350	50
73	L	900		N	1N	31	1N	1N	11	1N	385	50
75	L	300	N	N	1N	12	12	1N	11	31	668	50
75	L	400	N	N	1N	21	11	1N	11	11	350	50
75	L	500	N	N	1N	21	11	1N	11	11	363	50
75	L	600	N	N	1N	21	11	1N	11	11	301	50
75	L	700	N	N	1N	12	11	1N	11	11	288	50
75	L	800	N	N	1N	21	11	1N	11	31	544	50
75	L	900	N	N	1N	12	11	1N	11	11	467	50
77	L	300	N	N	1N	1N	1N	1N	1N	1N	767	50
77	L	400	N	N	1N	1N	1N	1N	1N	1N	347	50
77	L	500	N	N	1N	1N	1N	1N	1N	1N	363	50
77	L	600	N	N	1N	1N	1N	1N	1N	1N	342	50
77	L	700	N	N	1N	1N	1N	1N	1N	1N	325	50
77	L	800	N	N	1N	1N	1N	1N	1N	1N	629	50
77	L	900	N	N	1N	1N	1N	1N	1N	1N	471	50

TABLE 12 FLEXIBLE PAVEMENT VARIABLES, WASHINGTON DOT

Distress and Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Condition Survey Rutting, Cracking (Longitudinal and Transverse) Wave Sags, Raveling-Flushing (severity & extent)	Bump Count (PCA Roadmeter counts per mile)	Year
		ADT
	Bump Count Measurement Speed (mi/hr.)	Percent trucks
Patching (severity & extent)		

TABLE 13 RIGID PAVEMENT VARIABLES, WASHINGTON DOT

Distress and Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Condition Survey: Surface Wear, Blow-ups, Cracking, Raveling, Joint Spalling Pumping, Faulting-Warping (severity & extent)	Bump Count (PCA Roadmeter counts per mile)	Year
		ADT
	Bump Count Measurement Speed	Percent trucks
Patching (severity & extent)		


UTAH DEPARTMENT OF TRANSPORTATION
PAVEMENT EVALUATION SUMMARY

Project No ________ State Route 62 Section 3
County Piute District 3 Maintenance Shed 326
Section Limits: From X-Sec. chng. to X-Sec. chng.
Milepost 18.67 to 26.40 Length 7.73 Miles
Pavement Surface Type CARM

Figure 11 Example pavement condition history, Utah

```
PAVEMENT EVALUATION FOR STATE ROUTE 026   SECTION   6    SUB SECTION 0     SEVIER COUNTY (21)       DISTRICT 5    FAS-303
FROM SEVIER CO. LINE          MILEPOST  53.32          TO  JCT. SR-24 SALINA       MILEPOST  62.30          LENGTH    8.98
MATERIAL  COVER AGGREGATE ROADMIX (CMIX)               MAINTENANCE SHED 333        I.C. NO.  528            WIDTH  12.
YEARLY INCREASE IN 18K LOADS   5.0 %                   PRESENT 18K LOADS           1.71800+03               T.S.I.  2.0

       * * DYNAFLECT TEST DATA * *                              * * DYNAFLECT SUMMARY AND AVERAGE CONDITIONS * *
NO. OF TESTS  2     DATE  5/12/77  HR 12  MIN 30
TEMPERATURES:  AIR  71.00, SURFACE  77.00, PAVEMENT  75.00                                                 18K LOADS
WHL PATH OSWP     LANE EBL       LAST REVISION                          DMD         SCI          BCI       TO FAILURE     YITF
F=  .910                                                          MIN   .99         .24          .06       1.2502+06      17
                          DMD     SNSR 2   SNSR 3   SNSR 4   SNSR 5   MAX  1.32         .30          .18       4.9794+05      16
                                                                  AVE  1.15         .27          .12       7.6340+05      16
OUTLYING VALUES           ****     ****     ****     ****     ****
MEAN                      1.15     .88      .75      .62      .50     STRUCTURAL NO. REQUIRED FOR 10. YEARS ADDITIONAL LIFE IS 3.57
STANDARD DEVIATION        .23      .28      .34      .30      .22     AVERAGE SCI & BCI INDICATE PAVEMENT AND SUBGRADE STRONG.
VARIANCE                  .05      .08      .12      .09      .05     IF PRESENT TRENDS CONTINUE, THE STRUCTURAL NEEDS ARE
T(..)                     .71      .71      .71      .71      .71     LOW AND THE ROAD WILL PROBABLY LAST OVER TEN YEARS.

ACTUAL READINGS           .99      .69      .51      .41      .35     SCIREQ=  1.05  BCIREQ=   .24  DMDREQ=  3.16  1DSYRS= 13
                          1.32     1.08     .99      .84      .60

                                  * * SERVICEABILITY SUMMARY AND AVERAGE CONDITIONS * *
          NO. TESTS  2          DATE 11/29/77     MPH  50.                 PSI:  AVERAGE 3.5   MINIMUM 3.4   MAXIMUM 3.5

AVERAGE SURFACE WEAR  3.8           AVERAGE POPOUTS    4.               AVERAGE P.S.I. INDICATES THAT THE SERVICE NEEDS ARE MODE[illegible]
AVERAGE WEATHERING    4.0           AVERAGE UNIFORMITY 2.5              AND WILL PROBABLY FALL BELOW THE T.S.I. IN SIX TO TEN YE[illegible].
AVERAGE RUT DEPTH (IN)     .17      YH  9    YX 10    YA  9

        AVERAGE  CRACKING  PER  1000 SQ. FT.                                 AVERAGE                     AVERAGE CONDITION
   TRANSVERSE          LONGITUDINAL                LOAD                   PATCHING PER                  OF TRANSVERSE AND
SEALED    NOT       SEALED    NOT           MAP         ALLIG.             1000 SQ. FT.                    LONGITUDINAL
 (FT)    SEALED      (FT)    SEALED         TYPE        TYPE              SKIN      DEEP                      CRACKS
                                            SQ.FT.      SQ.FT.            SQ.FT.    SQ.FT.              OPEN.   ABRAS.   MULT.
  0.       0.          0.       0.          980.         0.                0.        0.                 4.0     4.5      4.2

       * * SKIDMETER TEST DATA * *                                      * * SKIDMETER SUMMARY AND AVERAGE CONDITIONS * *
NO. TESTS  5    DATE 11/15/77    TEMPS: AIR  51.00   ASPHALT  54.0
                                                                        SKID INDEX:  MINIMUM 54    MAXIMUM 67    AVERAGE 57
TEST      01   02   03   04   05   06   07   08   09   10   11   12   13
SKD IND   67   54   55   56   54   **   **   **   **   **   **   **   **      AVERAGE SKID INDEX INDICATES THAT THE ROAD IS
                                                                        MARGINAL, FURTHER MONITORING SUGGESTED.
```

Figure 12   Example pavement performance summary, Utah

TABLE 14 FLEXIBLE PAVEMENT VARIABLES, UTAH

Distress and Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Rut Depth(inches)	PSI	Year
Distress Index (dimensionless)		Accumulated Equivalent 18-kip Loads (latest year only)
Skid Index (Skid number)		
Deflections, Dynaflect and Surface Curvature Index (mils)		
Structural Index (dimensionless)		

provided summary sheets on over 50 pavement sections as well as copies of original data sheets for several sections.

An example summary sheet is shown in Figure 13 and a portion of the data record contained in this figure is reproduced in Table 15. Table 16 lists the paveemnt distress, performance and time variables recorded in the Minnesota data sample.

Illinois

After the completion of the AASHO Road Test, some test sections became part of U.S. 66 - Interstate 40. Though many sections were overlaid or reconstructed, a large number of the original rigid pavement sections from Loops 4, 5, and 6 remained in service. A small research staff remained at the test site and continued to record pavement condition and performance data for what is now referred to as the rehabilitated AASHO Test Road (Ref. 47)[1]. Mr. Don Schwartz of the Illinois DOT graciously provided summary sheets on sixty of the original rigid test sections and twenty sections constructed after the end of the Road Test. These sheets contain performance and condition records for the period 1962-1974, with sections identified by their original Road Test section number, where applicable.

Figure 14 illustrates a summary sheet for the rehabilitated AASHO Test Road, and Table 17 lists a portion of the data record. The pavement distress, performance and time variables involved are given in Table 18.

Georgia

Georgia has many miles of jointed concrete pavements. Faulting was recognized as a serious problem over a decade ago, and, consequently, extensive faulting surveys were conducted during the 1970s. Since faulting was identified as an important distress type for study in this project, and since other data sources did not provide adequate faulting records, ARE Inc requested a sample of faulting data from Georgia. Mr. Wouter Gulden of the Georgia DOT kindly supplied a summary of faulting in jointed concrete interstate pavements for the period 1971-1976, along with a graph relating faulting index value to cumulative truck traffic.

The pavement distress, performance and time variables included in the faulting summary are listed in Table 19.

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[1]Burke, J. and J.E. LaCroix, "Final Summary Report - Route U.S. 66 Condition Survey" (IHR-6), Research and Development Report No. 33, Illinois DOT, June 1971.

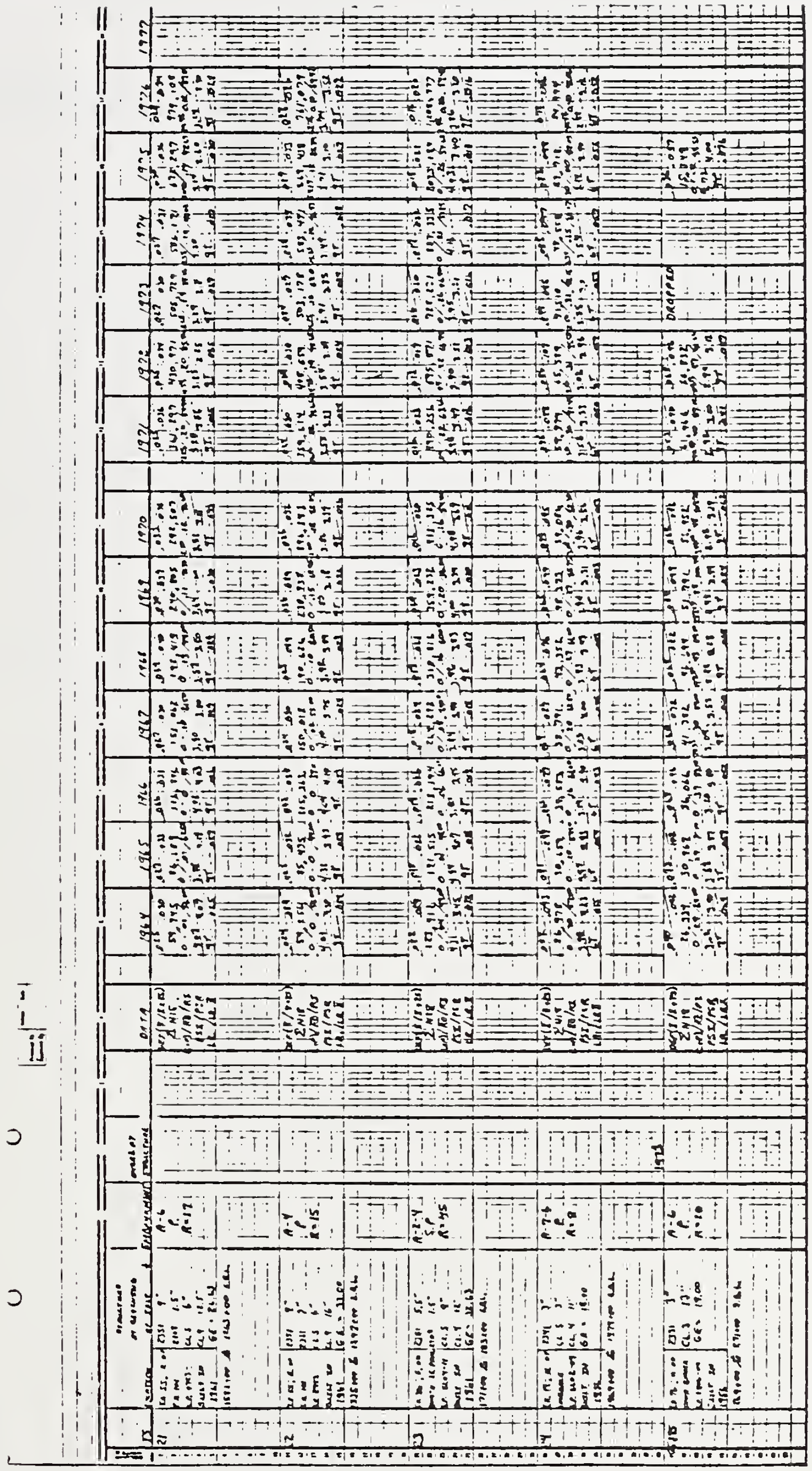

Figure 13 Example summary sheet, Minnesota

TABLE 15 PORTION OF DATA RECORD FROM SUMMARY SHEET, MINNESOTA

TS 21

T.H. 55 East of T.H. 101

S.P. 2723 - 1251 + 00 to 1263 + 00 E.B.L.

Built in 1961

2351 4 inches

2208 1.5 inches

CL. 5 6 inches

CL. 4 12.5 inches

G.E. = 26.63

A-6 P R-17

	1964	1965	1966	1967	1968
Deflection, $\bar{x}$	.025	.027	.026	.027	.029
$\bar{x} + 2\sigma$	.030	.033	.031	.030	.040
Σ N18	59,745	86,189	116,446	151,462	192,419
C + P	0	0	0	0	0
$\overline{RD}$	.01	.01	0	.10	.13
RI	56.00	62.50	58.00	61.50	74.00
PSI	4.07	3.88	4.01	3.90	3.57
PSR	4.07	4.17	4.03	3.80	3.50

1 inch = 2.54 cm.

TABLE 16 FLEXIBLE PAVEMENT VARIABLES, MINNESOTA

Distress and Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Rut Depth (inches)	PSI	Year
Cracking & Patching (sq.ft.per 1000 sq.ft.)	PSR (function of logarithm of counts per mile PCA Roadmeter)	Accumulated Equivalent 18 kip Axle Loads
Roughness Index (BPR Roughometer)		
Deflection (mils)		

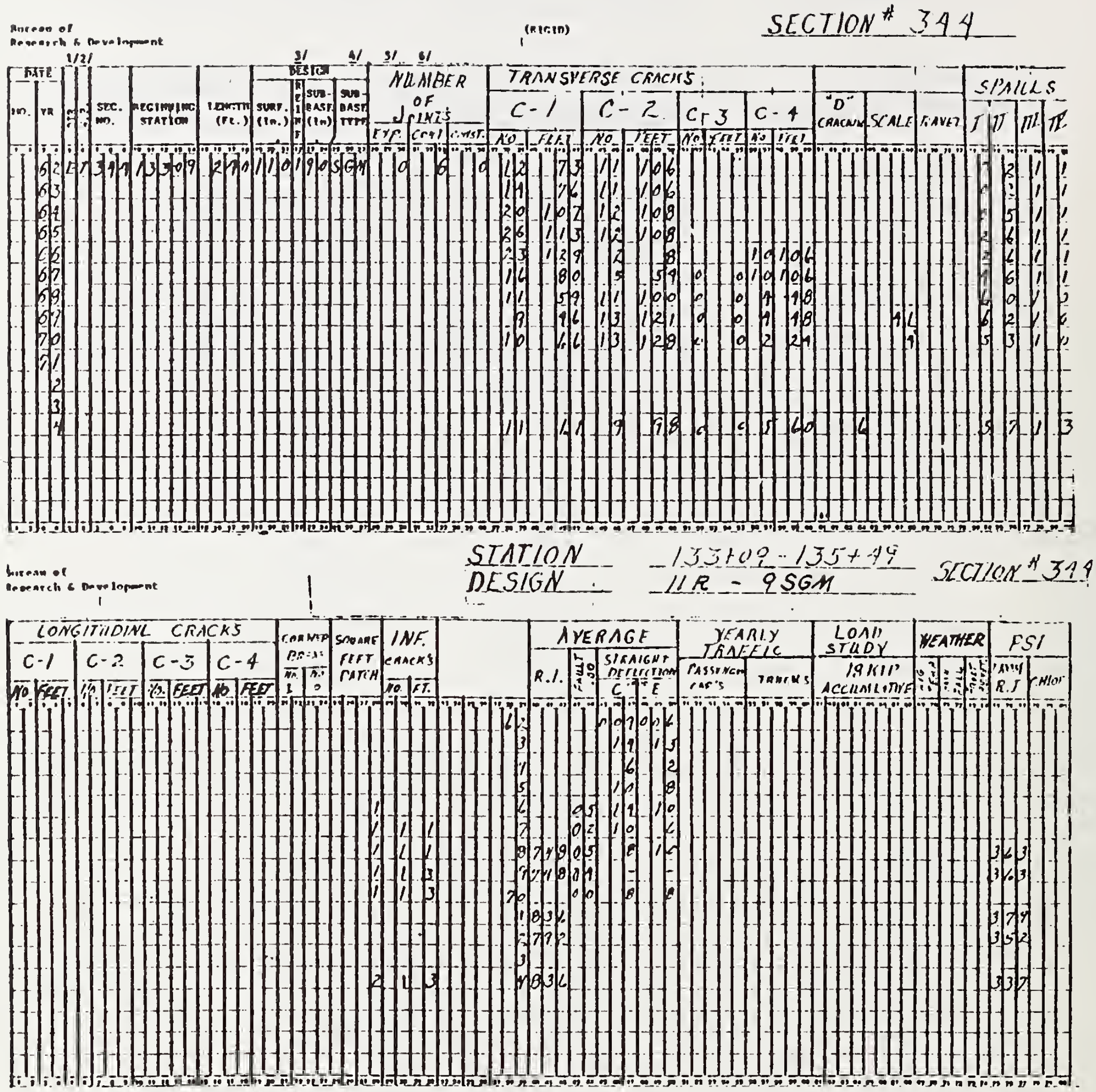

Figure 14 Sample summary sheet, Rehabilitated AASHO Test Road, Illinois

TABLE 17 PARTIAL DATA RECORD, REHABILITATED AASHO TEST ROAD, ILLINOIS

DIR	W.P.	SEC. NO.	BEGINNING STATION	LENGTH (Ft.)	SURF. (In.)	REINF.	SUBBASE (In.)	SUBBASE TYPE	NUMBER OF JOINTS EXP.	CONT.	CONST.
				DESIGN							
E	T	344	13309	240	110	1	90	SGM	0	6	0

DATE YR.	TRANSVERSE CRACKS C-1 No.	C-1 Ft.	C-2 No.	C-2 Ft.	C-3 No.	C-3 Ft.	C-4 No.	C-4 Ft.	"D" CRACKING	SCALE	RAVEL	SPALLS I	II	III	IV	LONGITUDINAL CRACKS	CORNER BREAK	SQUARE FEET PATCH
62	12	73	11	106								0	2	1	1	0	0	
63	14	76	11	106								0	2	1	1	0	0	
65	26	113	12	108								2	6	1	1	0	0	
66	23	129	2	8			10	106				2	6	1	1	0	0	1
68	11	59	11	100	0	0	4	48				6	0	1	0	0	0	1
69	9	46	13	121	0	0	4	48		41		6	2	1	0	0	0	1
72										4								
74	11	61	9	98	0	0	5	60	6			5	7	1	3	0	0	2

YR	INF. CRACKS No.	INF. CRACKS Ft.	AVERAGE R.I.	FAULT .00	STRAIGHT DEFL. .000 C	STRAIGHT DEFL. .000 E	LOAD STUDY 18 KIP ACCUMULATIVE	WEATHER AVG. TEMP.	WEATHER RAIN FALL	WEATHER FROST PENET.	PSI FROM R.I.	PSI CHLOE
62					009	006						
63					14	13						
65					10	8						
66				05	14	10						
68	1	1	748	05	8	10					363	
69	1	3	748	04	-	-					363	
72			792								352	
74	1	3	836								337	

1 in. = 2.54 cm. 1 ft. = 0.305 m.
1 sq. ft. = 0.093 sq. m. 18 kips = 80 kN

TABLE 18 RIGID PAVEMENT VARIABLES, REHABILITATED AASHO TEST ROAD, ILLINOIS

Distress or Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Cracking: Transverse, Class 1, 2, 3, and 4 Longitudinal, Class 1, 2, 3, and 4 (number and length) "D" Cracks (sq. ft.) Infiltration (number and length)	PSI (based on Roughness Index)	Year Accumulated Equivalent 18-kip Axle Loads
Sealing and Raveling (sq. ft.)		
Spalling Class 1,2,3,4 (number)		
Faulting (inches)		
Deflection, Center and Edge (mils)		

TABLE 19 RIGID PAVEMENT VARIABLES, GEORGIA

Distress or Related Variables	Serviceability or Performance Variables	Time or Traffic Variables
Faulting Index (accumulated faulting, five consecutive joints, thirty-seconds of an inch)	none available	Year

PUBLISHED REPORTS

In addition to the primary data sources described above, a number of research reports containing pavement condition and performance data are available in the literature. Three studies (Refs. 48[1], 49[2], and 50[3]) provided secondary data bases for use in this project.

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[1]Kallas, B.F. and J.F. Shook, "San Diego County Experimental Base Project, Final Report", Parts I and II, Research Report 77-1, The Asphalt Institute, November 1977.

[2]Shook, J.R. and B.F. Kallas, "Ordway, Colorado Experimental Base Project Performance Studies, Progress Report", Report No. CDOH-DTP-R-78-6, The Asphalt Institute, December 1978.

[3]Shah, G.N., "Performance Study of Continuously Reinforced Concrete Pavement on I-95", Report No. FHWA-MD-R-78-11, Final Report, June 1978.

CHAPTER 4. GENERAL DISTRESS TO PERFORMANCE RELATIONSHIPS

The distress/performance problem is quite complex. Consequently, the work undertaken in the early phases of the Project was directed toward the establishment of generalized distress/performance relationships and an identification of the primary, secondary, and time-dependent effects of pavement distress on serviceability and performance. The mechanisms and sequences of pavement distress occurrence were examined in terms of primary, secondary and time-dependent effects. A generalized distress sequence and some specific examples are presented below. Also presented are graphical representations of generalized relationships between various distress types and serviceability.

BASIC SEQUENCE OF PAVEMENT DISTRESS DEVELOPMENT

The sequence of occurrence of pavement distress varies among distress types and pavement types. However, the general features of distress development follow a basic pattern that has been identified in Reference 18. This basic sequence is illustrated in Figure 15.

The sequence proceeds as follows: An applied load or environmental effect initiates a response mechanism within the pavement, causing the pavement to exhibit a response. If this response exceeds some limiting value, a distress mechanism is initiated. This distress mechanism then produces, either immediately or after some repetition, a distress manifestation.

To illustrate this process, consider the development of fatigue cracking in flexible pavements. In this case, the response mechanism would be bending due to an applied wheel load. The primary response in tensile strain reaches or exceeds some limiting value. However, the distress mechanism is much more complex. The distress mechanism would be that process that reduces the resistance of the pavement material to tensile strains as a pavement is subjected to repetitive bending by passing wheel loads. As the pavement material is "fatigued" and its tensile strength is reduced by this process, the magnitude of the tensile strain response necessary to initiate a crack is continuously decreased. This process continues along with increased bending at the crack (and thus more tensile strain) while the crack propagates toward the surface. At some point as the crack approaches the surface, other distress mechanisms may come into play for the final development of the full-depth crack. It should be noted that the use of the term "fatigue" is simplistic since microcracks are generally presented in asphalt concrete materials. "Crack initiation" is more accurately described as enlargement of a microcrack, while crack propagation may be described as a continuous process of microcrack interconnections.

Figure 16 illustrates a slightly more complex version of the

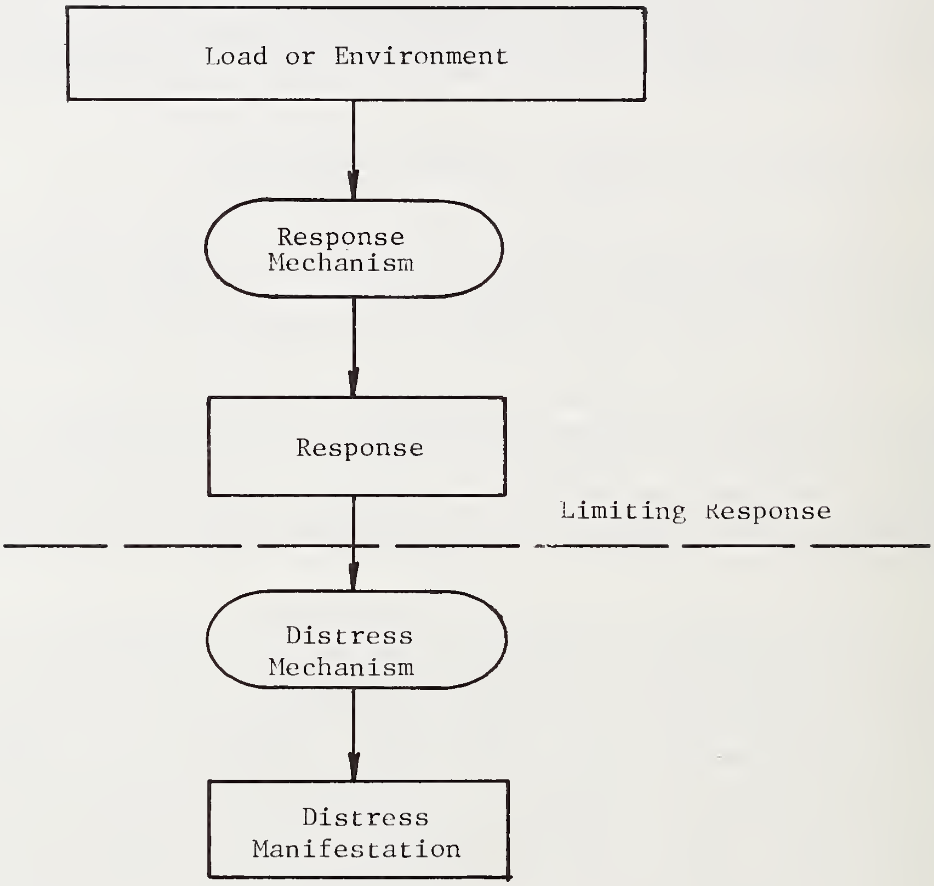


Figure 15 Basic distress sequence

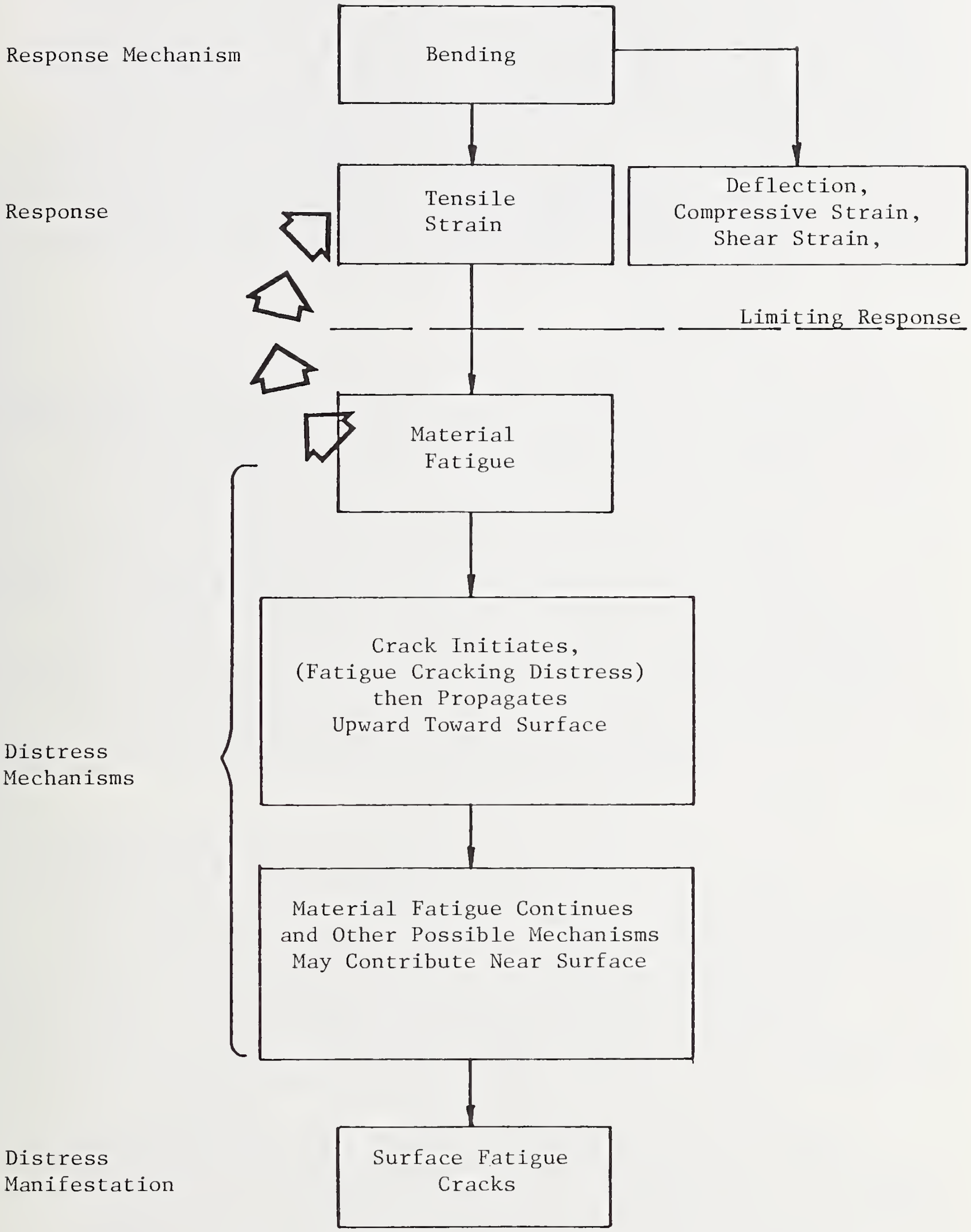


Figure 16 Development of fatigue cracking in an asphalt concrete pavement

development sequence for fatigue cracking. As before, the response mechanism is bending, but in this example the bending leads to the response tensile strain, deflection, compressive strain, and shear strain. The latter three do not have a direct effect and are shown to the side and terminating. The arrow from the distress mechanism is intended to show that it is active long before the limiting tensile strain is reached, and that it continues to lower the response threshold after the crack is initiated and contributes along with increased tensile strain to facilitate propagation of the crack. The distress in the form of a fatigue crack is shown to have occurred when a crack is initiated, but would manifest itself later as observable cracks at the surface of the pavement.

Secondary Distresses

One of the principle difficulties in establishing satisfactory relationships between distress and serviceability history or performance is the problem of incorporating secondary distresses in the relationships. For example, the occurrence of a moderate amount of surface cracking in a pavement has very little effect on riding comfort. However, cracking allows the infiltration of moisture into the base, subbase, and subgrade, which is one of the principle causes of subsequent loss of riding comfort. In effect, increased moisture creates a change in the pavement structure that causes it to respond differently to wheel loads than when the structure was uncracked. Figure 17 illustrates secondary distress due to fatigue cracking, beginning with the initial information of fatigue cracks. Moisture infiltrates the base, subbase, and subgrade through cracks and has the effect of altering the pavement material properties. The reduced stiffnesses allow more bending and increased tensile strain at new locations; and new cracks are initiated and propagate to the surface, where multiple cracks such as those often called "alligator cracking" result.

A second and parallel mechanism, pumping erosion, could also be initiated (Figure 18). In this case, the moisture infiltration reduces the moduli of elasticity for the base, subbase, and subgrade, while pumping erosion creates voids in the support under the surface layer. The secondary response mechanism is increased bending of the surface layer. The secondary responses are increased tensile strain and shear strain. The secondary distress mechanism is again material fatigue, but may include several distress mechanisms as discussed above. The secondary distress is again fatigue cracking, but it has been accelerated and the eventual result will be multiple or alligator cracking.

A similar visual outline could be drawn using both the effects of moisture infiltration, pumping, and the resultant accelerated fatigue cracking to follow through to rutting and formation of "pot holes" as other secondary or tertiary distresses. These last two distresses affect the serviceability directly and create a need for maintenance.

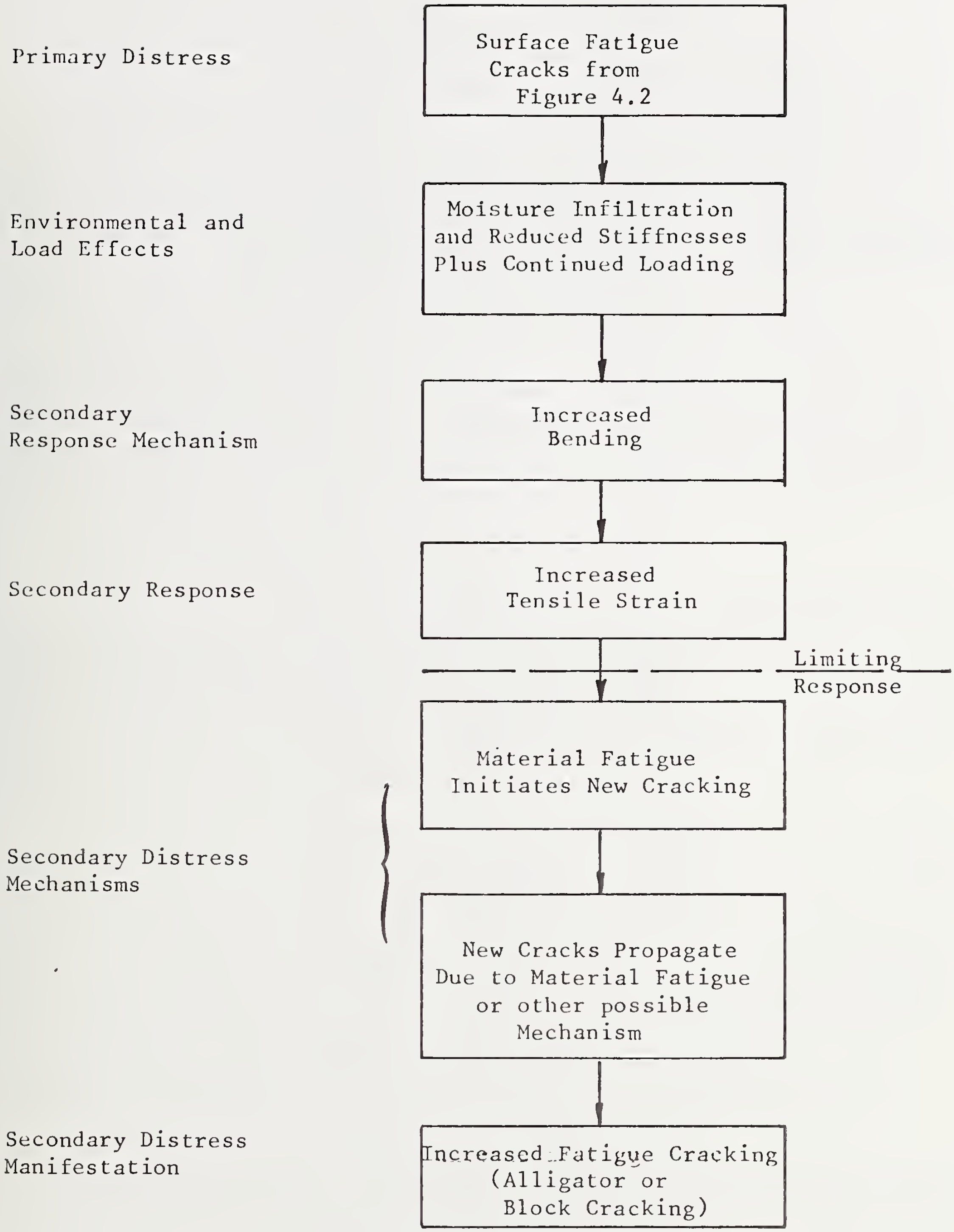


Figure 17 Increased fatigue cracking (secondary distress) in an asphalt concrete pavement due to moisture infiltration

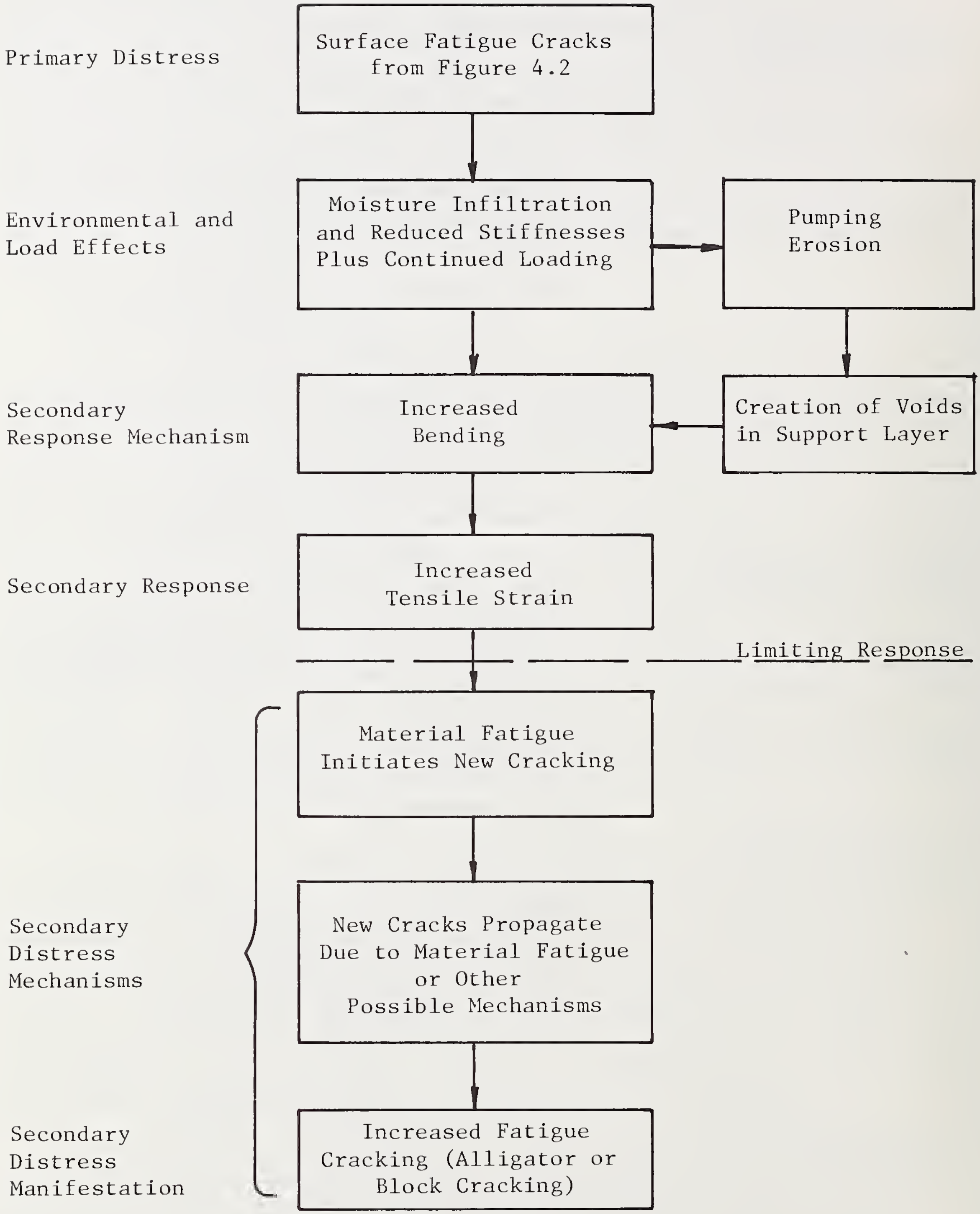


Figure 18 Increased fatigue cracking (secondary distress) in an asphalt concrete pavement due to pumping erosion

The basic sequence of distress development discussed above is applied in expanded form to several selected distress types in Appendix D.

DISTRESS AND SERVICEABILITY PATTERNS

The sequences of distress development in the previous section and in Appendix D provide a basis for distress/performance modeling. The next step is to include serviceability along with the distress variables, and consider the development of these variables as a function of time or accumulated loads. In this manner, distress and serviceability patterns were developed by the project Advisory Panel. This panel was composed of experienced engineers from design, maintenance and research divisions of state highway agencies as well as pavement researchers.

No universal agreement on distress and serviceability patterns was achieved. Thus, although the sequence of distress development is basically the same in all areas of the country, the details of the distress and serviceability patterns with time are by no means identical. Nevertheless, a number of distress/serviceability patterns were identified, and these provide a reasonable illustration of the range of patterns found throughout the nation.

Flexible Pavement

Figure 19 is a composite drawing illustrating three typical patterns for distress and serviceability in asphalt concrete pavements as a function of accumulated load applications. These curves are abstracted from the general shapes of fifteen different distress and serviceability histories, used for a variety of designs and conditions, drawn by different individuals. An example of these histories for a range of pavement designs and environments is given in Figure 20.

In addition, the relationship between the occurrence of a specific distress and the consequent loss of serviceability is expected to depend on which of the major distress modes develops first. In many situations, rutting is the first type of distress to be observed in a flexible pavement. During the first 2-3 years it would not be unusual to measure average rut depths in the range of 1/8" (0.32 cm.) to 1/4" (0.64 cm.). The cause of this early rutting is thought to be associated primarily with densification of the asphalt concrete or upper layers of the pavement structure. Some plastic flow may also occur; however, in most situations this will not be progressive; in fact, it would be very difficult to identify the relative proportation of densification and plastic flow for small magnitudes of rutting.

As materials densify they appear to approach a steady state; i.e., very little additional rutting per load application.

Asphalt concrete can probably accommodate an average level of

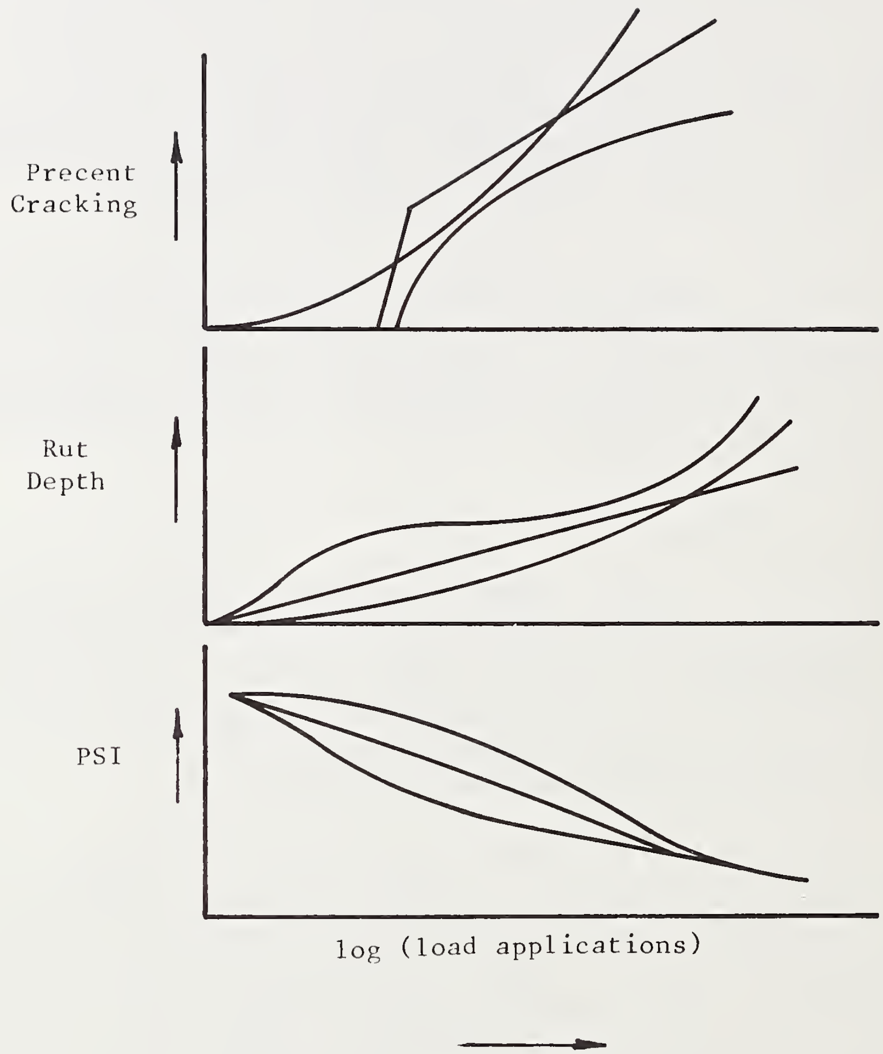
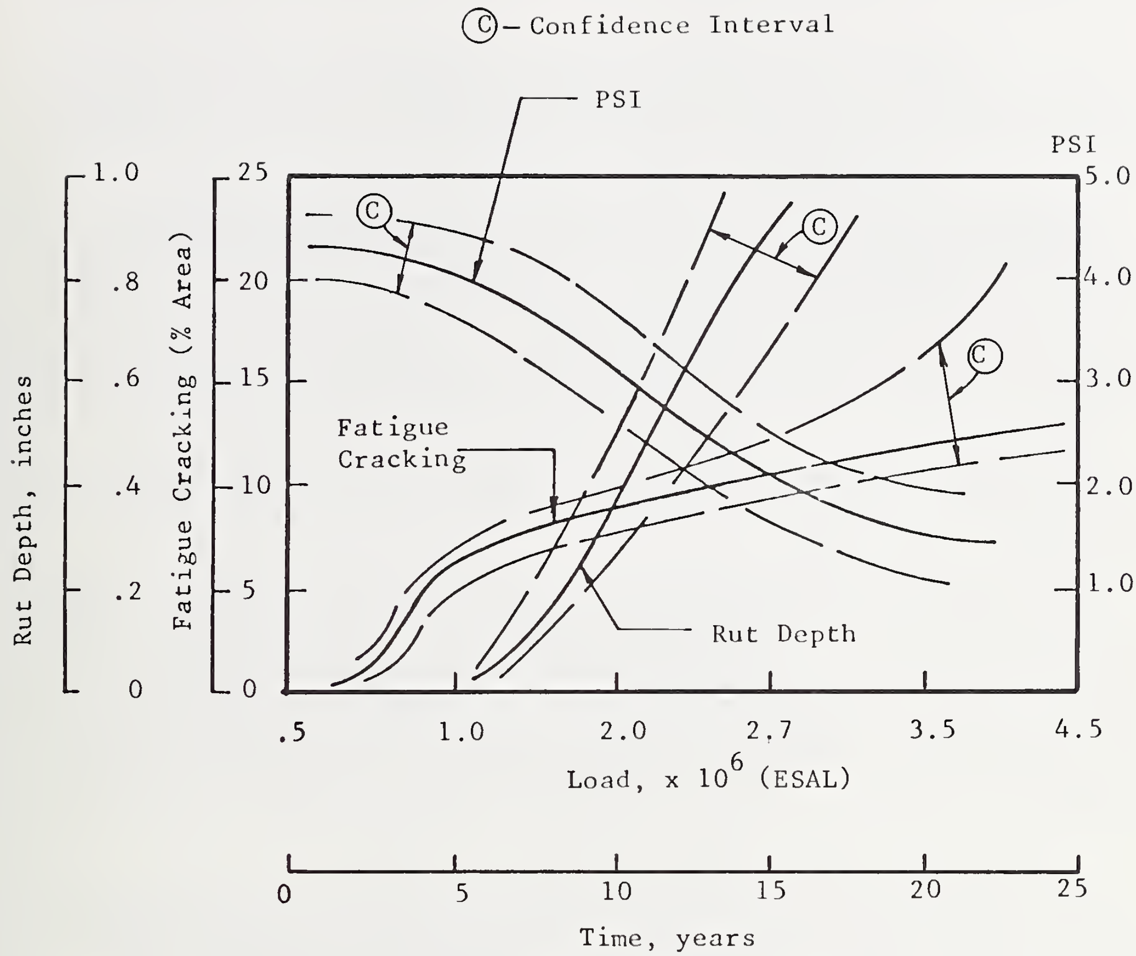


Figure 19 Typical distress and serviceability history patterns, flexible pavements



Notes: Low Temperature Cracking - not anticipated
Fatigue Cracking in percent of wheel path area
Asphalt Concreet: 4 to 6 inches, Dense graded asphalt
Aggregate base and subbase: 20 to 25 inches
Subgrade: Fair to poor
Averave conditions: $\leq$ 20 inches rain, MAAT 60°F (16°C)
Frost penetration - not significant
Drainage - included in design, No swelling soils

1 inch = 2.54 cm.

Figure 20 Flexible Pavement History

rutting of 3/8" (1 cm.) without seriously increasing the potential for cracking; when rutting exceeds this amount, longitudinal cracking may occur along the edge of the rut.

As the asphalt ages and becomes more brittle the potential for cracking increases. In most cases some cracking in the rutted area can be expected to occur after 6-8 years of service. If rutting reaches 3/4" (2 cm.), cracking could occur sooner.

When cracking occurs in a pavement exhibiting rutting greater than 3/4" (2 cm.) it will probably cause the rate of rutting to increase which in turn will accelerate the progression of cracking. Ruts form channels in which water can be stored and cracks provide an easy path for the water to enter the substructure. The result would be an increase in the level of deflection, under traffic, with possible "pumping of unbound fines in the aggregate base", as discussed previously.

A possible distress/serviceability history for an asphaltic concrete pavement section with rutting as the primary distress is shown in Figure 21.

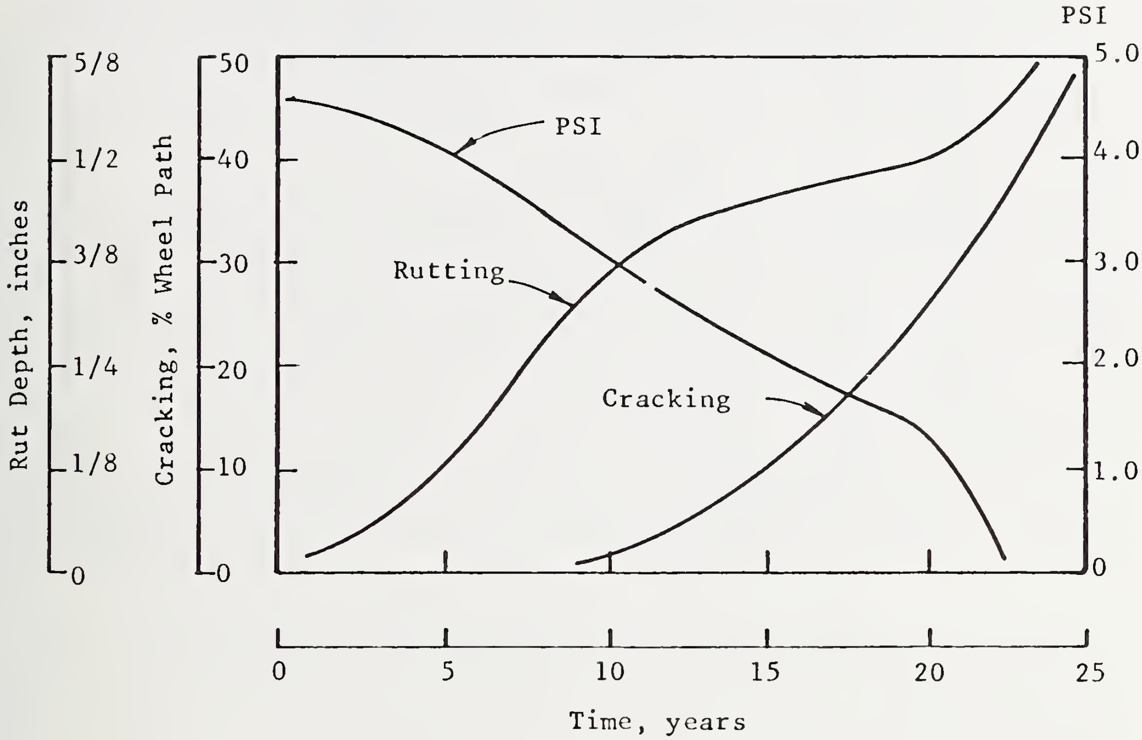
The other possibility of major interest is that premature fatigue cracking may occur. If fatigue cracking occurs first, or at a somewhat accelerated rate, it could hasten the time at which pavements would exhibit significant levels of rutting. For example, in most situations rutting will occur first and cracking may not be observed during the first 6-8 years of service. However, if for some reason the cracking starts in 2-3 years it could accelerate the early stages of rutting by 2-3 years.

The mechanism in this case is similar to that for rutting as the primary distress mode; i.e., the pavements leak water to the substructure causing pumping and deflection which will most probably lead to rutting. This mechanism is discussed in detail in the proceeding section.

Premature fatigue cracking is considered more damaging to the life cycle of the pavement than is premature rutting, assuming rutting is not dominated by plastic flow or shear deformation.

Figure 22 illustrates a possible distress/serviceability history with fatigue cracking as the primary distress. Notice that rutting nevertheless plays a significant role in this case as well. This reflects the predominant opinion of the advisory panel regarding the relative impact of rutting and fatigue cracking on serviceability. That is, in developing Figures 21 and 22, the following criteria were used:

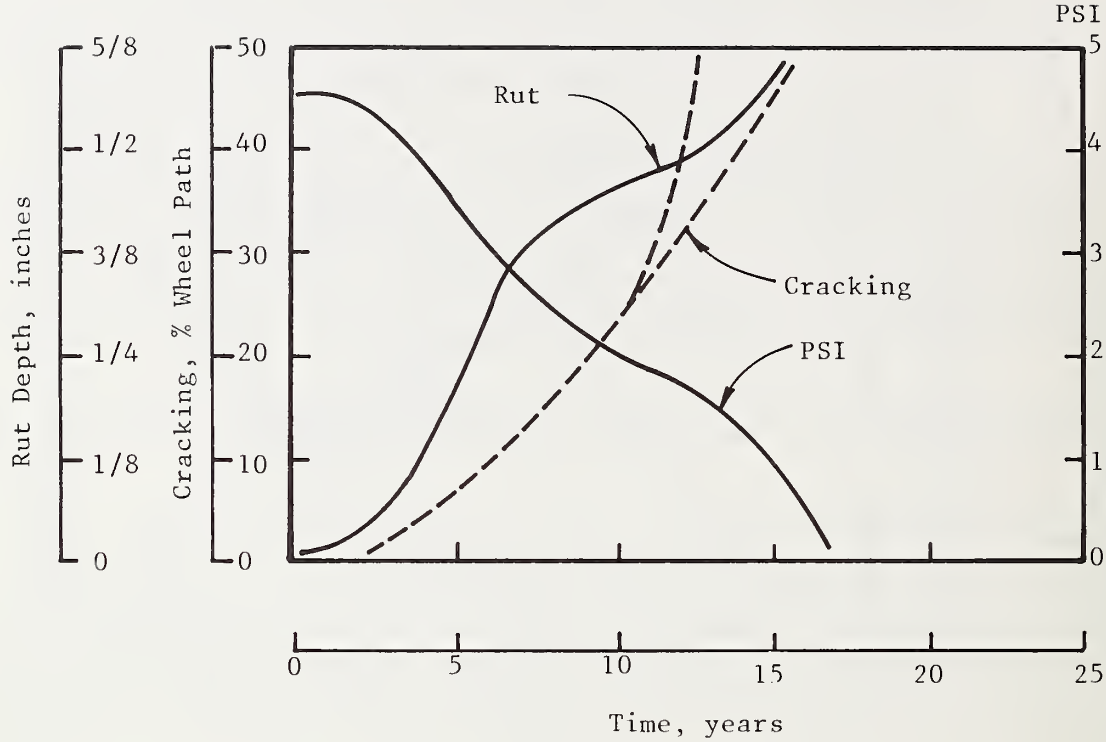
(1) PSI is responsive to the amount of rutting and the rate of rutting.



Notes: A.C - 4 to 6 inches
Average conditions
Environment: $\leq$ 20 inches of rain
MAAT 60°F (16°C)
Frost penetration - not significant
No unusual subsoil or drainage conditions

1 inch = 2.54 cm.

Figure 21 Flexible pavement distress/serviceability history with rutting as the primary distress



Note: Average conditions (See Figure 21)

1 inch = 2.54 cm.

Figure 22 Flexible pavement distress/serviceability history with fatigue cracking as the primary distress

(2) PSI is not as immediately responsive to cracking as it is to rutting; however, the occurrence of cracks would be expected to have some effect on PSI both present and future.

Rigid Pavements

Figure 23 illustrates some typical distress and serviceability patterns for portland cement concrete pavements as a function of time. Again, these curves are composites abstracted from sketches drawn by several different individuals. An example of these individual drawings is provided in Figure 24.

The sequence of distress for rigid pavements will be a function of the basic geometry of the individual pavement slabs. In general, low temperature and shrinkage cracks will not occur in the shorter pavement slabs. However, once the slab length exceeds approximately 30 feet (9.1 m.), low temperature or shrinkage cracking will probably occur intermediately between construction joints. This crack will usually occur at a section that has been weakened by a cut and should not present an unsightly crack at the surface. The load transfer at such a crack must occur from aggregate interlock. If aggregate interlock does not occur or if particularly expansive aggregates are used, the crack may open excessively, permit entry of foreign material, then local overstressing of the concrete produces subsequent surface spalling problems and potential blowups at a later time.

In jointed concrete pavements, the next potential type of distress to occur is the vertical differential elevation that results in either step faulting or pumping, or a combination of the two. Step faulting is generally more of a problem in areas where adequate moisture for pumping is not available. Such faulting is a result of repetitive loading which produces a kneading action of the subbase materials that produces an accumulation of material under the upstream slab. The upstream slab is actually lifted above the level of the downstream slab producing the step faulting phenomenom. In cases where free moisture is available, deflections at the pavement edge, joint, or crack can produce hydrostatic forces that will eject water and materials at these discontinuities. It should be emphasized that free water must be available at the interface between the bottom of the pavement surface and the top of the subbase for pumping to occur. Once pumping occurs and a void is present beneath the pavement surface, fatigue cracking can occur if excess tensile stresses occur in the PCC slab.

In addition to the types of distresses mentioned above, D-cracking can occur in portland cement concrete pavement structures when aggregates that are particularly temperature and moisture susceptible are used in the concrete mix.

This type of distress occurs primarily in the mid-western part of

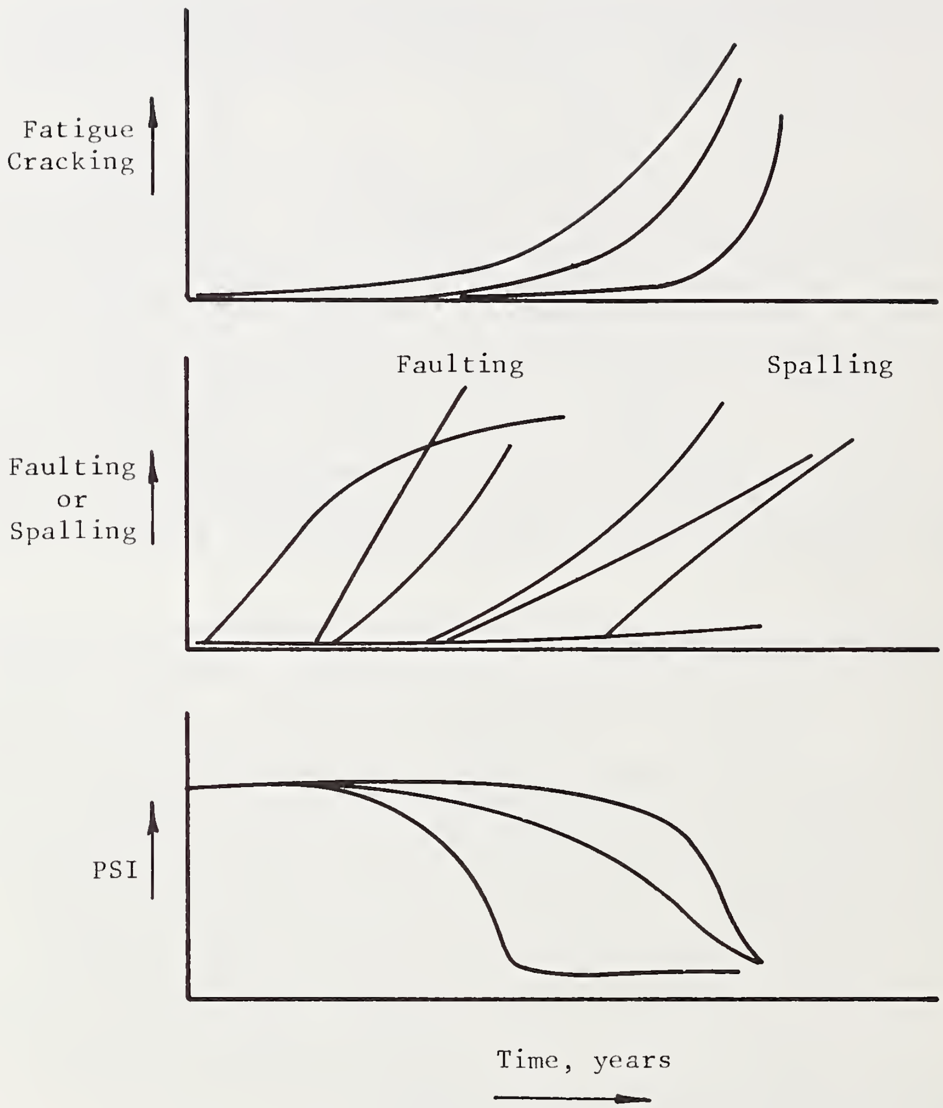
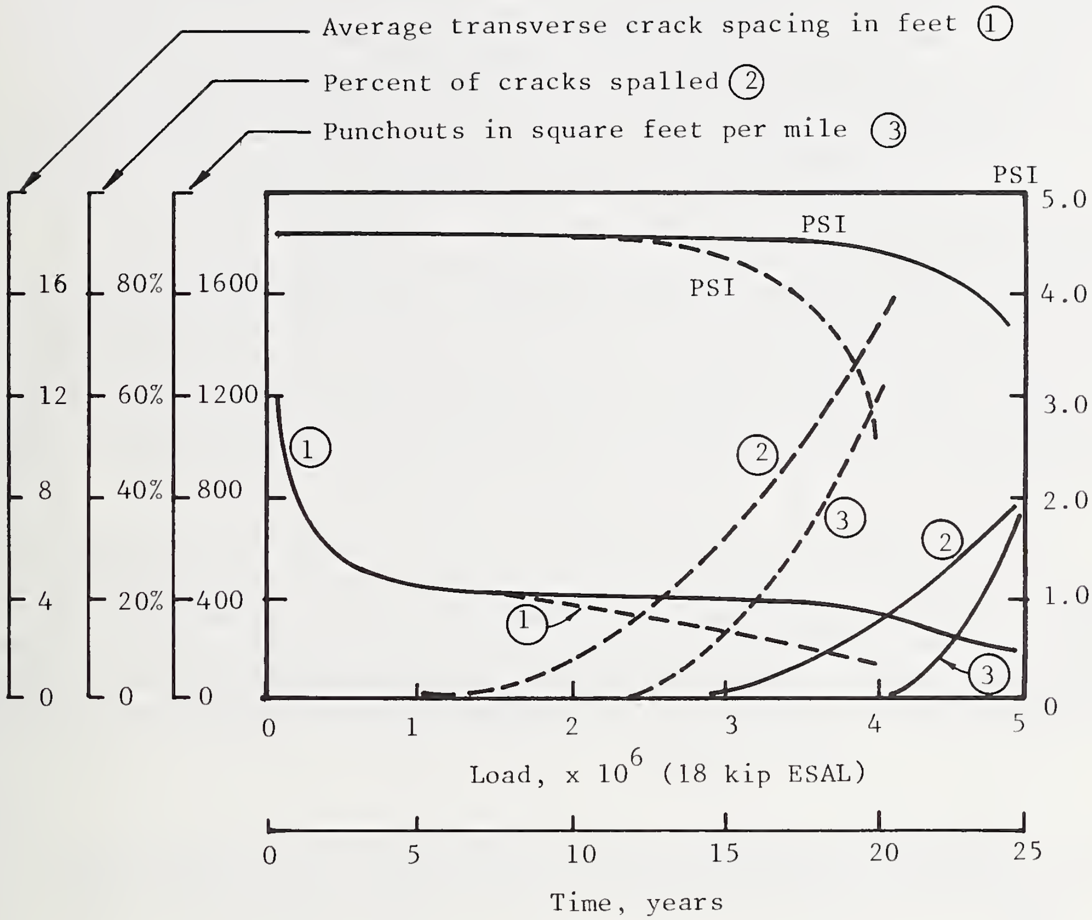


Figure 23 Typical distress and serviceability history patterns, rigid pavements



Notes: Transverse Cracking { Low Temp/Shrinkage = 1st few years of curve; Fatigue = Load-associated tail of curve }

Solid lines represent "high" performance
Dashed lines represent "low" performance
9" PCC slab with welded wire mesh reinforcement (0.7% steel)
18" open graded subbase; good drainage ditches
Sandy loam soil (5') over limestone
Central Texas climate - 25" rain and 10 freeze/thaw annually
Routine maintenance performed at 3/4 desired level due to budget restrictions

	18 kips = 80 kN
1 foot = 0.305 m.	1 mile = 1.6 km.
1 inch = 2.54 cm.	

Figure 24 Rigid pavement history, CRCP

the United States. In these areas marginal aggregates are the only ones available for use in the portland cement concrete. When these marginal aggregates are used in environments that experience a large number of freeze-thaw cycles the integrity of the concrete mass may be affected. There are a number of studies (Refs. 42[1], 43[2], 44 through 50, 51[3], 52[4], 53[5], 54[6], 55[7], 56[8], 57[9], and 58[10]) that indicate conflicting factors that affect the occurrence of the D-cracking in concrete pavements. However, there seems to be agreement that the problem is accentuated by the occurrence of deicing salts used in maintenance operations. These aggregates are generally absorptive and the water accumulates in the larger aggregates of the concrete mixture, the water freezes, and the large aggregates are shattered. Following the fracturing of these aggregates, the integrity of the concrete slab is compromised, additional water accumulates in the cracks and voids and further deterioration occurs. The distress is evidenced at the surface when a series of crescent shaped cracks occur along transverse and longitudinal joints and cracks. The manifestations of D-cracking do not usually appear until 5 years, and in some cases 10 years, into the service life of the structure.

The distresses mentioned above can occur in any type of concrete pavement. There are two types of distress that occur in continuously reinforced concrete pavements that should be mentioned separately from those occurring in the other two types of concrete pavements. These two types of distresses are deep crack spalling and punchouts. Deep crack spalling occurs as a result of the low temperature and shrinkage cracks that develop, as shown in Appendix D. When cracks that have a spacing

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[1]"Asphalt Concrete Overlays of Flexible Pavements, Vol. 1, Development of New Design Criteria", ARE Inc, Report No. FHWA-RD-75-76, June 1975.

[2]Barker, E.S., "Calculation of Maximum Pavement Temperatures from Weather Reports", Highway Research Board Bulletin 168, 1957.

[3]Smeaton, K.W., "An Interactive Approach to Developing Pavement Distress-Performance Relationships", A Thesis presented to The University of Waterloo, Waterloo, Ontario, 1978.

[4]Havens, J.H., "The D-Cracking Phenomenon: A Case Study for Pavement Rehabilitation", Kentucky Department of Transportation, Lexington, Kentucky, April 1976.

[5]Bukovatz, J.E., C.F. Crumpton, and H.E. Worley, "Kansas Concrete Pavement Performance as Related to D-Cracking", Transportation Research Record 525, 1974.

greater than 8 feet (2.5 m.) are subjected to repetitive wheel loads, deep spalling can occur. When the crack spacing is less than 8 feet, condition surveys in Texas by McCullough have shown that deep spalling does not occur. With crack spacings greater than 8 feet (2.5 m.), large crack openings occur and excessive vertical deflections are sustained by the pavement structure. When these vertical deflections occur, the top half of two adjacent slabs is placed in compression and a shear fracture of the concrete in adjacent slabs occurs. The depth of the spall is normally about one-half the depth of the concrete slab and extends upward at a 45 degree angle to the surface.

Crack spacing in CRCP is critical in development of distress. Not only does deep spalling result from crack spacing, but if the crack spacing is less than 4.4 feet (1.3 m.), punchouts can occur. When repeated wheel loads are applied to these pavement slabs, the load transfer deteriorates and the pavement slabs begin to respond to load as transverse beams rather than longitudinal beams. The fatigue mechanism described in Appendix D becomes active and closely spaced longitudinal cracks occur within the transverse beam section. These small blocks then are free to rotate and move and will eventually move vertically downward and punchout.

The development of punchouts and deep crack spalls in continuously reinforced concrete pavements occurs at variable times and is more a function of the geometry of the slab and the traffic loading, than the age of the pavement structure itself.

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[6]Missouri State Highway Department, "Investigation of D-Cracking in PCC Pavements", Missouri State Highway Department, Jefferson City, Missouri, Phase I, May 1971, Phase II, March 1972, and Phase III, January 1977.

[7]Verbeck, G., P. Klieger, S. Start, and W. Teske, "Interim Report on D-Cracking of Concrete Pavements in Ohio", Portland Cement Association, Skokie, Illinois, March 1972.

[8]Williams, F.M., A. Trefny, J.T. Paxton, and H.D. Davis, "Development of Laboratory Methods for Determining D-Cracking Susceptibility of Ohio Gravel and Limestone Coarse Aggregates in Concrete Pavements", Ohio Department of Transportation, Columbus, Ohio, February 1974.

[9]Missouri State Highway Department, "Investigation of Roadway Design Variables to Reduce D-Cracking", Missouri State Highway Department, Jefferson City, Missouri, October 1975.

[10]Best, C.H., "D-Cracking in PCC Pavements: Cause and Prevention", National Technical Information Service, Springfield, Virginia, June 1974.

CHAPTER 5. DATA ANALYSIS AND MODELING

The primary modeling effort for this project involved three separate, but related investigations:

(1) Verification of the distress and performance patterns (Chapter 4) by comparison with objective data.

(2) Development of time-independent distress/serviceability relationships.

(3) Development of time-dependent distress/serviceability relationships.

Models were developed on the basis of both subjective and objective data, as discussed below.

VERIFICATION OF DISTRESS AND PERFORMANCE PATTERNS

The patterns of distress and serviceability presented in Chapter 4 were developed on the basis of subjective data, i.e., expert opinion. An attempt was made to verify these patterns through comparison to objective data. Portions of the data records described in Chapter 3 were used for this purpose.

Flexible Pavement Patterns

A sample of forty flexible pavement performance records was selected from the AASHO Road Test data for the purpose of verifying distress and serviceability patterns. These sections were chosen randomly, subject to two requirements:

(1) The pavement remained in service for at least half of the test period.

(2) The pavement reached a terminal PSI on or before the end of the test.

The first requirement was necessary to insure that a reasonable time base was involved, while the second was necessary to allow an investigation of the shape of the serviceability history curve for lower values of PSI.

The AASHO Road Test data records show PSI plotted as a function of time. Hence, the most direct comparison was possible with Figure 20 or similar figures. The general shape of the PSI curves in such figures was indeed found in several of the AASHO Test Sections. However, a

large number of sections exhibited so much variation in serviceability that no clearly discernable pattern could be established.

In the hope of achieving better verification, several of these sections were replotted as a function of the logarithm of the accumulated load. The resulting patterns were compared with those of Figure 19. Again, some of the pavement sections fit the predicted patterns, but the variation was so large as to prohibit verification of any specific pattern.

These results are not directly attributable to any specific factors. However, it is likely that the accelerated nature of the testing at the AASHO Road Test could have a significant influence on the serviceability patterns of the test sections.

Twenty-four test sections from the Minnesota data base were also used to investigate serviceability and distress patterns. Plots of serviceability as a function of load and time were generated, and comparisons were made to Figures 19 and 20. In this case, the patterns in Figure 19 were more satisfactorily verified, with the two upper curves in Figure 19 accounting for over 60% of the pavement sections. However, variation again made the task of recognizing patterns somewhat difficult.

The principal pattern observed in the plots of serviceability as a function of time was a linear decrease in serviceability with time, as illustrated in Figure 25. This is very similar to the central section of the serviceability curves illustrated in Figure 20 and hence, may be taken as partial verification of the figure patterns. However, variation again played a significant role. In addition, several sections exhibited an increasing serviceability trend as a function of time.

As mentioned in Chapter 4, Minnesota records have two serviceability variables: "PSI" and "PSR". The former is calculated from an AASHO-type equation involving both roughness and distress, while the latter is calculated directly from roughness measurements. Both of these serviceability measures were used in the pattern analysis described above, and the results were quite similar in each case.

Fatigue cracking patterns could not be identified in either the AASHO or Minnesota data base. The primary reason is that both Minnesota and the AASHO Road Test report only the summary statistic cracking plus patching. Plots of cracking plus patching versus time were made for both the AASHO and Minnesota data. Several of the pavement sections in each case followed the general exponential rise of cracking shown in Figure 20. However, no specific pattern could be identified as dominant, since considerable variability was observed.

Rutting patterns, on the other hand, were more pronounced. The

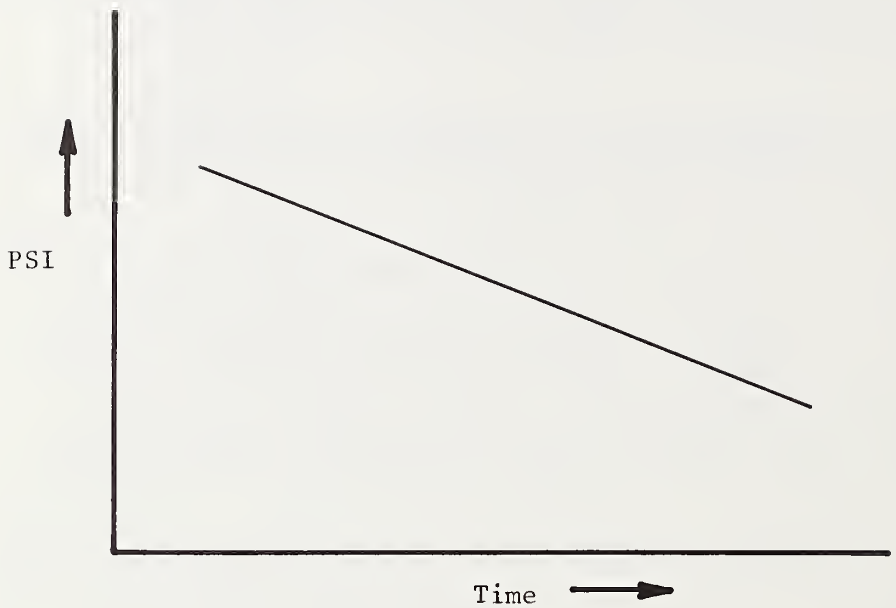

Figure 25 Primary pattern of serviceability history, Minnesota

principal trend observed in both the AASHO and Minnesota data bases was that of Figure 20. Thus, the rapid onset of rutting followed by a slow increase over the years, as illustrated in Figure 20, can be considered as more firmly established than any other distress or serviceability pattern for flexible pavements. However, several of the Minnesota sections exhibited linear trends in rut depth, and a few "scattergun" patterns were observed.

Rigid Pavement Patterns

AASHO Road Test data were also employed in an attempt to verify the rigid pavement distress and serviceability patterns illustrated in Figure 23. As before, only those sections which survived for more than half of the test period and which reached a terminal serviceability before the end of the test were considered. Thirty-five test sections meeting these criteria were selected for this purpose.

The serviceability history patterns observed in these sections as a function of time were the most consistent, repeated patterns of any distress or serviceability variable for either flexible or rigid pavements. Approximately 90% of these pavements exhibited one of the upper two curves in Figure 23, and two-thirds of the sections showed the characteristic drop-off of the upper-most curve in Figure 23. There was no apparent difference in the shape of the serviceability history curve for reinforced and unreinforced pavements. Thus, the serviceability patterns of Figure 23 are considered verified.

Faulting and spalling are not included in the AASHO Road Test data base. A pumping score is included, which is somewhat related to faulting, and many of the class 3 and 4 cracks were undoubtedly spalled. However, it is not possible to extract specific faulting and spalling information from the AASHO data. Similarly, none of the other data sources described in Chapter 3 are suitable for verification of faulting of spalling patterns. Those data bases which include faulting or spalling information fall into two general categories:

(1) An insufficient number of data points are included to establish a pattern.

(2) The distress is measured in terms of severity and extent classifications, rather than actual numerical values.

Thus, it was not possible to verify the patterns of Figure 23 for faulting of spalling. However, it should be mentioned that faulting patterns similar to those shown in Figure 23 have been observed by other investigators (Ref. 61)[1]. Also, the Georgia faulting data described in Chapter

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[1]MacLeod, D.R., "Considerations for Maintenance Strategies for Portland Cement Concrete Pavements", Thesis, University of California, Berkeley, 1979.

3 are reasonably consistent with these faulting patterns.

Similarly, fatigue cracking was not reported in the AASHO data base. However, class 3 and 4 cracks may be considered as including the category fatigue cracking. The general pattern of major cracking (Class 3 and 4) for the 35 selected AASHO Test sections reflects the exponential growth of fatigue cracking shown in Figure 23. Once again, though, considerable variation is observed in the patterns for individual pavement sections.

TIME INDEPENDENT DISTRESS/SERVICEABILITY RELATIONSHIPS

Model equations for serviceability as a function of distress, without regard to time dependence, were developed for the flexible and rigid pavement. Those data sources were used which were considered to offer the best potential for useful model building in each case.

Flexible Pavement Models

Several analyses were conducted to determine possible relationships between serviceability and the high priority distress variables selected in Chapter 2.

Rutting. Graphs of "PSR" and "PSI" as a function of rut depth were prepared for the combined 50 test sections of the Minnesota data base. These plots are presented here as Figure 26 and 27, respectively.

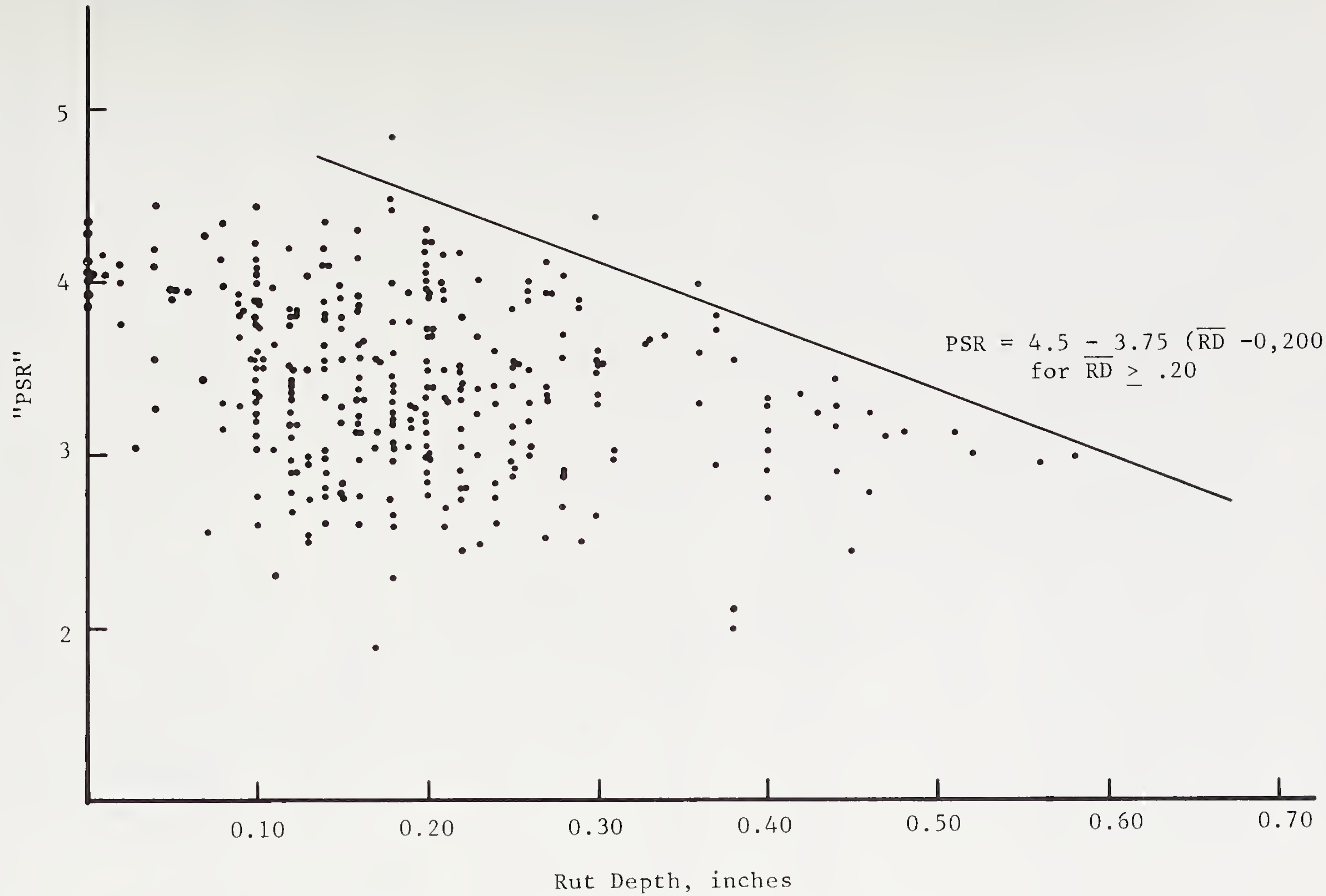
The original objective in generating these plots was to develop an equation for PSI or PSR as a function of rut depth which would be valid for all of the pavements in the Minnesota data base. However, as can be seen in Figures 26 and 27, a considerable amount of variation was encountered. That is to say, any model fit to these data by regression would of necessity exhibit a poor fit (low value of R^2). However, after examination of these plots, it was clear that another approach could be taken. It is easily observed in these figures that for any given rut depth, there is a maximum value of PSR or PSI. In Figure 26, for example, the maximum value of PSR falls on or about the line PSR = 4.5 for rut depths less than approximately 0.2 inches (0.5 cm.). For rut depths larger than 0.2 inches (0.5 cm.), the maximum PSR values fall on or near the line:

$$PSR = 4.50 - 3.75\ (\overline{RD} - 0.20) \qquad (3)$$

Based on these observations, two hypothesis were put forward:

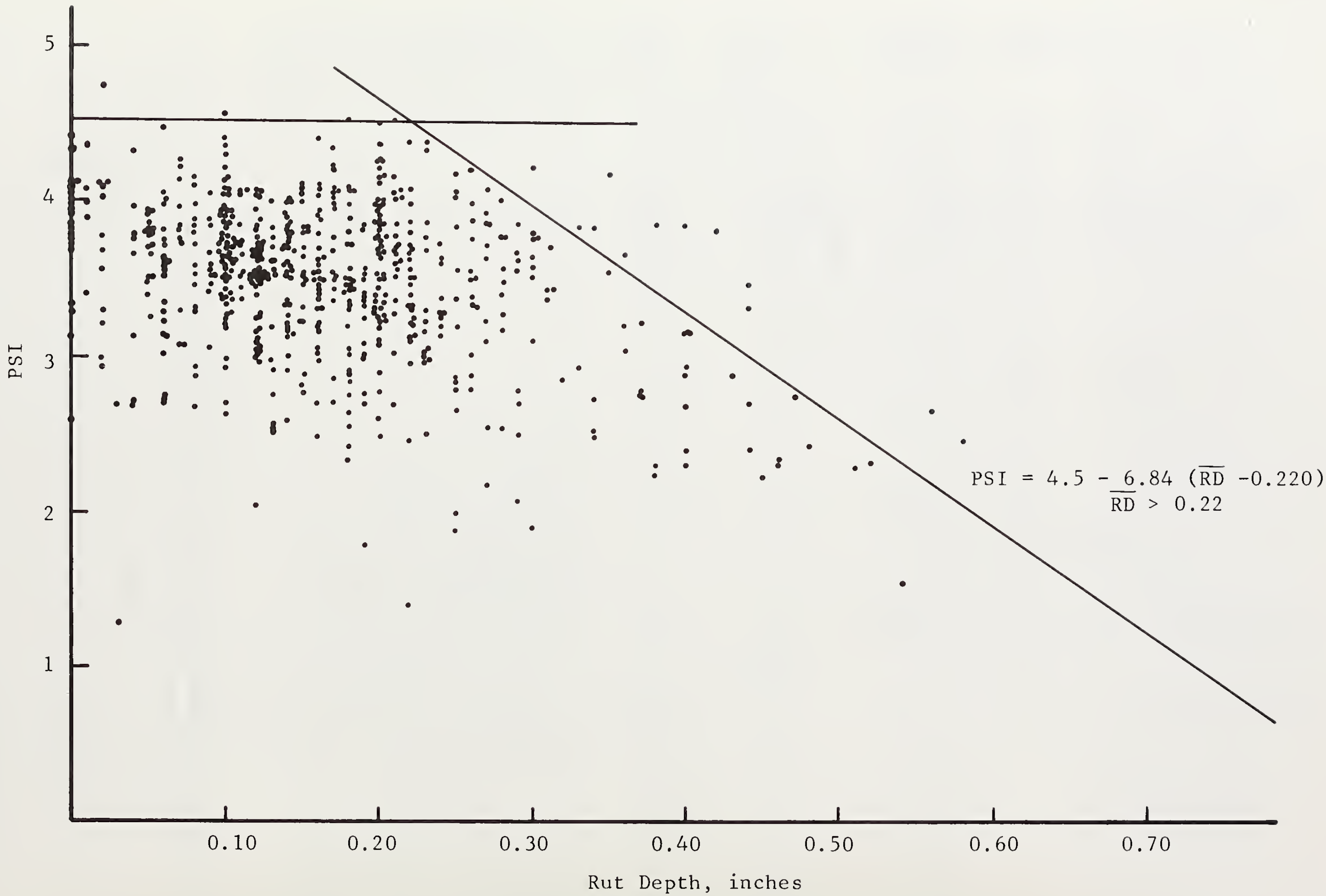
(1) Rut depths less than 0.2 inches (0.5 cm.) have no affect on PSR, and
(2) For rut depths greater than 0.2 inches (0.5 cm.), the minimum effect of rutting on PSR is given by equation (3).

Additional losses in serviceability due to other distresses, environ-



1 inch = 2.54 cm.

Figure 26 "PSR" as a function of rutting, Minnesota



1 inch = 2.54 cm.

Figure 27 "PSI" as a function of rutting, Minnesota

mental effects, etc. are then assumed to produce the distribution of PSR values shown in Figure 26.

Attempts were made to verify these hypotheses by identifying those pavement sections exhibiting very small amounts of cracking plus patching, the only other distress variable included in the Minnesota data. Those pavements exhibiting cracking plus patching scores less than 5 sq. ft. (0.5 sq. m.) per 1,000 sq. ft. (93 sq. m.) were found scattered throughout the distribution of data points in Figure 26, however. Thus, it was not possible to confirm these hypotheses. On the other hand, it was not possible to rule out these hypotheses, since other major variables of interest were not included in the data base.

A similar analysis based on Figure 27 leads to the equation:

$$PSI = 4.50 - 6.84\ (\overline{RD} - 0.22) \qquad (4)$$

Again, the same results were found. In fact, "PSR" and "PSI" are highly correlated, as shown in Figure 28, although considerable individual variations in "PSR" and "PSI" values for specific pavement sections are observed.

Plots of PSR and PSI versus rut depth were subsequently generated for pavement sections with various ranges of cracking plus patching. In all cases, considerable variations occurred, such that it was not possible to confidently establish a trend. A typical case, using only those pavements with cracking plus patching greater than 50 sq. ft. (4.6 sq. m.) per 1,000 sq. ft. (93 sq. m.), is shown in Figure 29. In general, the variation was not as pronounced for pavements exhibiting higher values of cracking; however, very few pavement sections exhibited cracking plus patching exceeding 100 sq. ft. (9.3 sq. m.) per 1,000 sq. ft. (93 sq. m.) of pavement, so that further investigation was inhibited.

An investigation of the rut depth data from Utah indicated that only three pavement sections experienced rut depths greater than one quarter inch. Even these sections showed no correlation between rut depth and serviceability.

Several of the Brampton Test Road sections exhibited significant rutting, but again, no correlation with serviceability (Riding Comfort, on a 0-10 scale) was found, other than a slight trend toward decreasing serviceability with increasing rut depth. Figure 30 shows the Brampton rut depth - serviceability data. Note that this figure is reasonably consistent with the "pattern" in Figures 26 and 27.

Fatigue Cracking. Of the data sources discussed in Chapter 3, only the Washington and TTI data bases contain a separate record of fatigue cracking (referred to as "alligator cracking"). As will be discussed in Chapter 6, the analysis of data from these sources is complicated by the fact that the record is in terms of severity and extent categories,

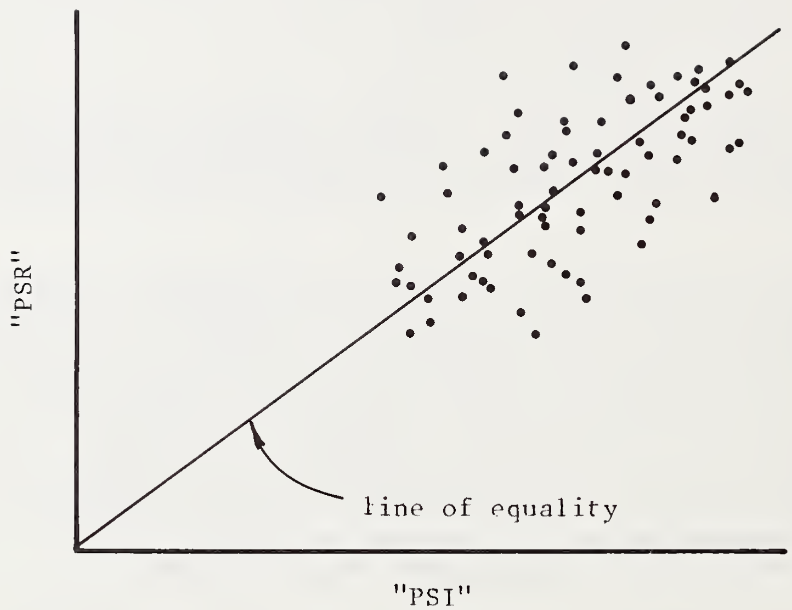


Figure 28 "PSR" versus "PSI", Minnesota

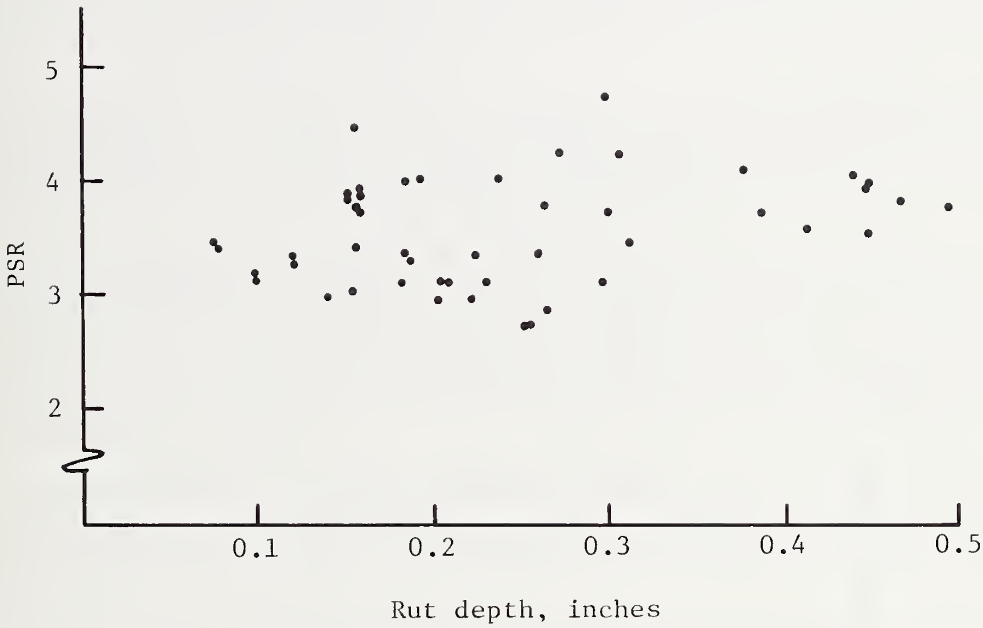


Figure 29 "PSR" versus rut depth for those sections with C + P > 50, Minnesota

1 inch = 2.54 cm.

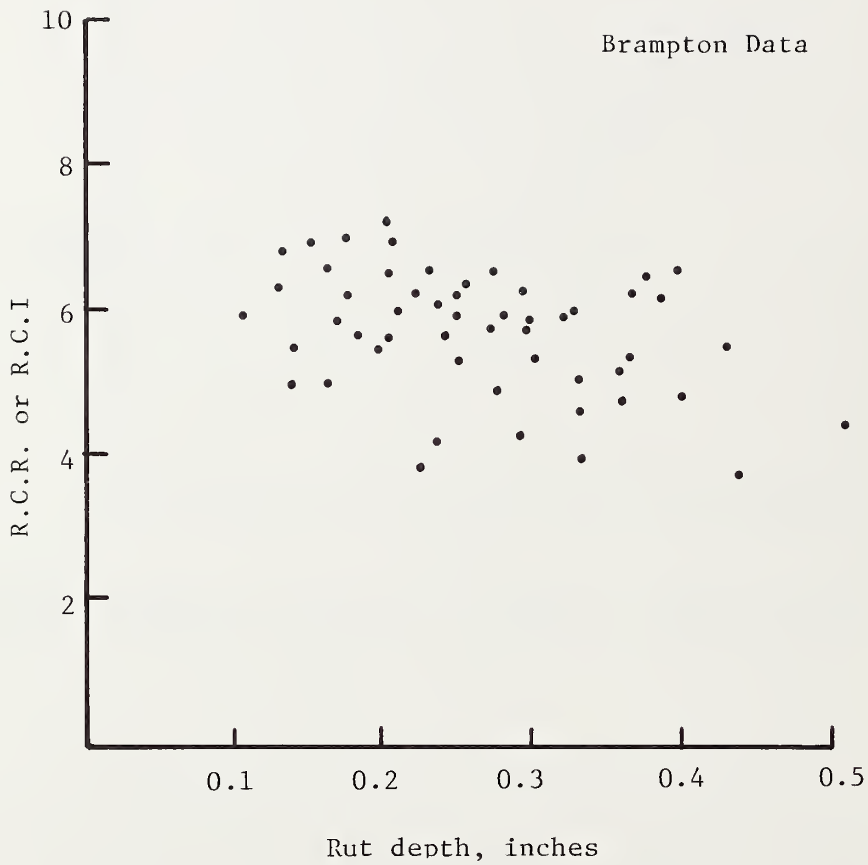


Figure 30 Riding comfort versus rut depth, Brampton

1 inch = 2.54 cm.

rather than actual numerical values. However, plots of serviceability and distress variables were made for twenty-five test sections randomly selected from the TTI data base. Visual examination of these graphs failed to produce any relationship between the occurrence of fatigue cracking and serviceability. In addition, histograms of alligator cracking and "bump count" were produced by computer from the Washington data base. For each combination of severity and extent, the number of pavements within a given range of "bump count" were plotted, as shown in Figure 31. A simple statistical analysis, including mean and standard deviations, was also conducted. In general, the mean bump count increased with increasing severity and extent of alligator cracking, but no specific relationships were found.

The Ordway, Colorado research study also included "alligator cracking", measured in sq. ft. per 1,000 sq. ft. (93 sq. m.). However, only four pavement sections exhibited significant fatigue (greater than 40 sq. ft. (3.7 sq. m.) per 1,000 sq. ft. (93 sq. m.)), hence, no meaningful analysis was possible.

The Minnesota variable C + P (cracking + patching) includes only block or alligator cracking, but is of course confounded with patching. Nevertheless, this variable was found to be reasonably well correlated with serviceability, as discussed in a subsequent section.

Low-Temperature Cracking. None of the data sources for this project include low-temperature cracking records. The Washington, TTI and Brampton data bases do record "transverse cracks", which may reasonably be assumed to be largely low-temperature cracks. However, in each case, data are recorded in terms of severity and extent levels, rather than actual numerical values.

As in the case of fatigue cracking, sample plots of the TTI data and histograms of the entire Washington data base were examined. No viable relationship between the occurrence of transverse cracks and serviceability was found.

General Distress/Serviceability Relationships. Multiple regression analyses were performed on the combined data records from the fifty test sections in the Minnesota data base. Attempts were made to generate equations for both "PSR" and "PSI" as a function of various combinations of rut depth and cracking plus patching. The variables listed in Table 20 were used in the stepwise multiple regression program STEP01 to produce twenty-seven equations for PSI and PSR. The results may be summarized as follows:

(1) The variable which was most highly correlated with both PSI and PSR, and which was in each case the first variable selected by STEP01 to enter the regression, was $\sqrt{C+P}$, where C+P = cracking plus patching in sq. ft. per 1,000 sq. ft. (93 sq. m.).

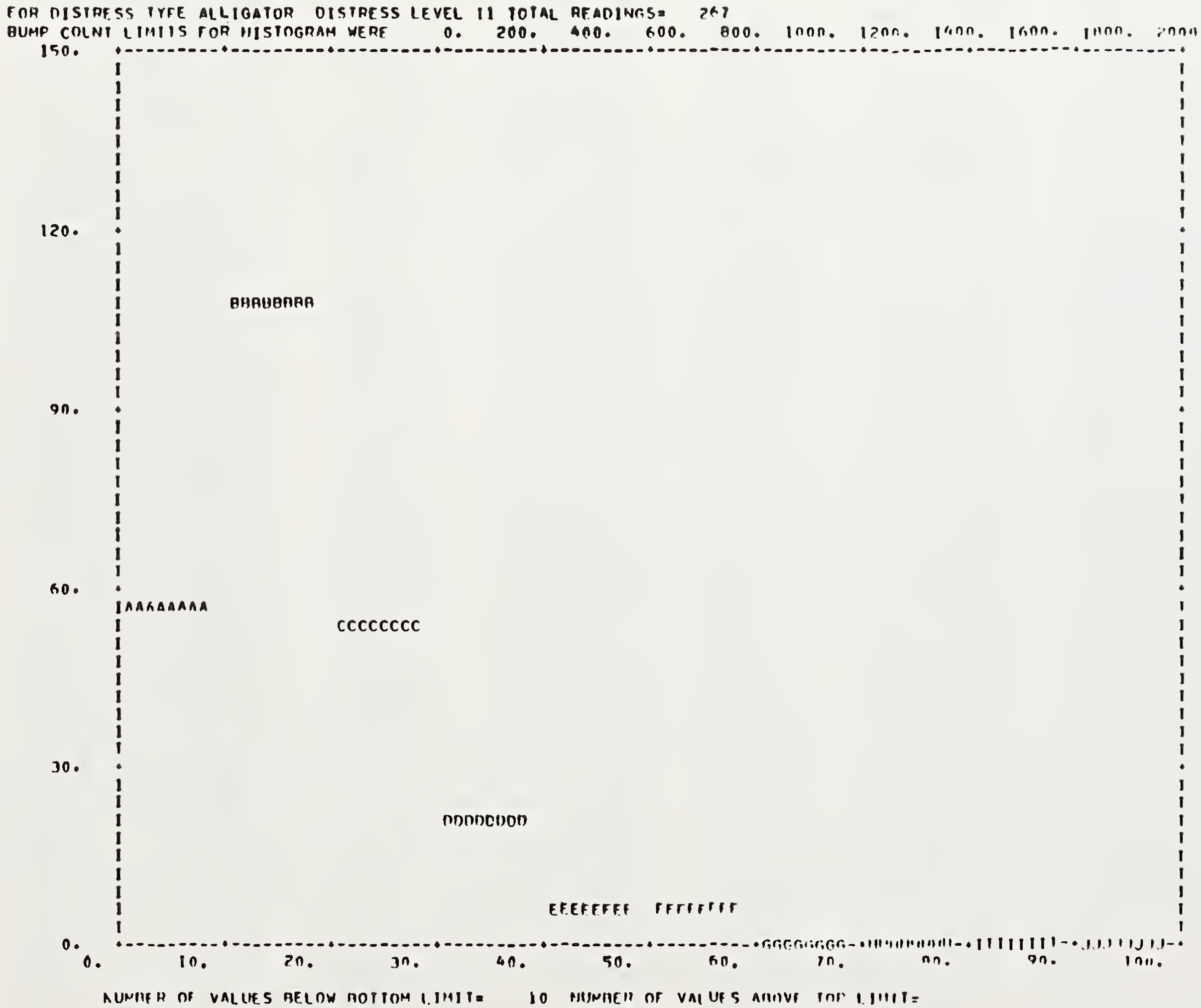


Figure 31 Bump count levels for alligator cracking of severity 1 and extent 1, Washington

TABLE 20 TRANSFORMED VARIABLES FOR MULTIPLE REGRESSION, MINNESOTA

(1)	PSI or PSR
(2)	RD (rut depth)
(3)	C + P (cracking + patching)
(4)	RD^2
(5)	$(C + P)^2$
(6)	RD^3
(7)	$(C + P)^3$
(8)	$RD \cdot (C + P)$
(9)	$RD \cdot (C + P)^2$
(10)	$RD^2 \cdot (C + P)$
(11)	$(RD - 0.20)$
(12)	$(C + P - 20.0)$
(13)	$\sqrt{RD}$
(14)	$\sqrt{C + P}$
(15)	e^{RD}
(16)	$e^{(C + P)}$

(2) The addition of other distress variables did not significantly improve the fit obtained with $\sqrt{C+P}$ alone. The order of variables entered and the corresponding values of R^2 are listed in Tables 21 and 22. As can be seen in these tables, the use of up to twelve variables in the regression improved R^2 by less than 0.04 over the value obtained with $\sqrt{C+P}$ alone.

(3) The relatively low values of R^2 (≈ 0.4) are unfortunate, but not unexpected. The variability observed in Figures 26 and 27 is indicative of a large "pure" error, and the plots of residuals for each of the regression variables are fairly randomly scattered. An exception is found in the residuals plots for PSI and PSR, which show a strong linear trend. The explanation for this may lie in the fact that both PSI and PSR are defined by equations which include roughness, but roughness was not included in the regression. Had roughness been included, a "perfect" fit could have been obtained by reproducing the equations which were used to calculate the recorded PSI and PSR values.

Based on these results, the most valid equations found for the full set of Minnesota data are:

$$PSI = 3.65 - 0.0786 \sqrt{C+P} \quad (5)$$

with $R^2 = 0.379$, and

$$PSR = 3.83 - 0.0785 \sqrt{C+P} - 0.491 \sqrt{RD} \quad (6)$$

with $R^2 = 0.372$.

Only one independent variable is included in the PSI equation, because the second variable entered in the PSI regression was another function of (C+P), and only a very small improvement in R^2 was involved.

Further analysis was carried out in an attempt to identify subsets of the data for which more meaningful models could be constructed. Sections with large traffic volumes showed no more reliable distress/serviceability patterns than did pavement sections with low traffic volumes. An exception to this general rule was found in one section, believed to be a grain route, which carried a much larger traffic volume than any other section in the study. However, it was not felt that a single section could provide a basis for generalization.

Finally, the effect of pavement structure on distress/serviceability relationships was investigated. The fifty test sections were divided into three categories based on the Gravel Equivalent (G.E.) numbers recorded in the data base. Category 1 involved G.E. less than 18, Category 2 involved G.E. greater than 18, but less than 25, while Category 3 involved G.E. exceeding 25. Multiple regression analysis was run

TABLE 21 RESULTS OF STEPWISE REGRESSION, MINNESOTA, PSI EQUATION

VARIABLE ENTERED	VARIABLE REMOVED	# VARIABLES IN EQUATION	R^2
$\sqrt{C + P}$	-	1	0.379
$(C + P)^2$	-	2	0.389
$\sqrt{RD}$	-	3	0.399
$RD \cdot (C + P)^2$	-	4	0.400
RD^3	-	5	0.400
$e^{(C + P)}$	-	6	0.401
$(C + P - 20.0)$	-	7	0.405
$RD \cdot (C + P)$	-	8	0.406
RD^2	-	9	0.406
$RD^2 \cdot (C + P)$	-	10	0.408
e^{RD}	-	11	0.408
-	$\sqrt{RD}$	10	0.408
$(RD - 0.20)$	-	11	0.408
$\sqrt{RD}$	-	12	0.408

TABLE 22 RESULTS OF STEPWISE REGRESSION, MINNESOTA, PSR EQUATION

VARIABLE ENTERED	VARIABLE REMOVED	# VARIABLES IN EQUATION	R^2
$\sqrt{C + P}$	-	1	0.357
$\sqrt{RD}$	-	2	0.372
$RD \cdot (C + P)^2$	-	3	0.380
$(RD - 0.20)$	-	4	0.382
$(C + P)$	-	5	0.382
$(C + P - 20.0)$	-	6	0.385
$(C + P)^2$	-	7	0.387
$RD \cdot (C + P)$	-	8	0.393
$RD^2 \cdot (C + P)$	-	9	0.395
$(C + P)^3$	-	10	0.396
RD^2	-	11	0.396
RD^3	-	12	0.397
RD	-	13	0.397

for each category separately, using the same variables listed in Table 20 in the program STEP01. The results of these regressions are summarized in Tables 23 and 24. The results may be summarized as follows:

(1) Again, the variable which was most highly correlated with PSI was $\sqrt{C+P}$. This variable was also highly correlated with PSR, but another variable, $\sqrt{RD}$, was found to be more highly correlated to PSR in Gravel Equivalent classes 1 and 3.

(2) The addition of other distress variables did significantly improve the fit. The improvement was most noticeable in Gravel Equivalent Class 3, and for the PSR equation in all classes.

(3) Again, a linear trend was observed in the plot of residuals for PSI and PSR. As before, this may be attributed to the omission of roughness data from the regression.

(4) The higher the Gravel Equivalent class, the higher the value of R^2 obtained in all cases. This is particularly noticeable for the PSR equation, where extremely poor fits were obtained for Classes 1 and 2 while reasonably good fit was obtained for Class 3.

Since the equation defining PSI involves distress and roughness, while the PSR definition involves only roughness, it is not surprising that in general better fits were obtained for the PSI equation. However, for Gravel Equivalent Class 3, which includes pavements with Gravel Equivalents exceeding 25, the multiple regression analysis showed that the distress variables were more strongly correlated to PSR than to PSI. This indicates that for thicker, stronger pavements, cracking, patching and rutting show good correlation with roughness; while for weaker pavements there is virtually no correlation. It is possible that for weaker pavements some of the more "minor" distress types, such as ravelling, corrugation, and shoving, form a major component of pavement roughness, whereas for stronger pavements, these distresses are not significant. Since no other distress variables were recorded in the Minnesota data, it was not possible to test this hypothesis.

The improved equations, corresponding to equations (5) and (6), but generated for pavements with G.E. exceeding 25, are:

$$PSR = 4.21 - 0.162 \sqrt{C+P} + 0.00847 (C+P) \quad (7)$$

with $R^2 = 0.529$, and

$$PSI = 4.04 - 0.0827 \sqrt{C+P} - 1.46 \sqrt{RD} \quad (8)$$

with $R^2 = 0.461$.

TABLE 23 RESULTS OF STEPWISE REGRESSION BY GRAVEL EQUIVALENT CLASS, MINNESOTA, PSI EQUATION

CLASS 1 G.E. < 18
CLASS 2 $18 \leq$ G.E. 25
CLASS 3 G.E. ≥ 25

G.E. CLASS	NO. VARIABLES IN EQUATION	R^2
1	1	.386
	2	.398
	4	.407
	6	.422
	8	.432
	11	.436
2	1	.413
	2	.432
	3	.459
	4	.463
	5	.474
	7	.485
	13	.489
3	1	.353
	2	.461
	3	.474
	4	.496
	5	.523
	7	.567
	8	.573
	11	.581

TABLE 24 RESULTS OF STEPWISE REGRESSION BY GRAVEL EQUIVALENT CLASS, MINNESOTA, PSR EQUATION

CLASS 1 G.E. $<$ 18
CLASS 2 18 $\leq$ G.E. $<$ 25
CLASS 3 G.E. $\geq$ 25

G.E.CLASS	NO. VARIABLES IN EQUATION	R^2
1	1	0.076
	2	0.088
	3	0.106
	5	0.125
	9	0.145
	12	0.146
2	1	0.141
	2	0.186
	3	0.220
	4	0.236
	8	0.240
	11	0.242
3	1	0.382
	2	0.452
	3	0.529
	4	0.558
	5	0.574
	6	0.585
	8	0.606
	10	0.608

Rigid Pavement Models

Relationships were sought between serviceability and the higher-priority distress variables selected in Chapter 2. However, considerable difficulty was encountered in finding data records for these distress types.

Cracking. None of the data bases described in Chapter 3 includes fatigue, low-temperature or shrinkage cracking as separate categories. The AASHO Road Test data reports four classes of cracking, based on the size of the crack. Classes 3 and 4 (major cracking) may be considered to be associated with fatigue or low-temperature cracking. The Illinois data from the Rehabilitated AASHO Test Road include both transverse and longitudinal cracking classified into the same four classes. Washington reports a single cracking variable by severity and extent.

About one-third of the 85 test sections of the Rehabilitated AASHO data base exhibited appreciable major transverse cracking (classes 3 and 4). No major longitudinal cracking was found. A half-dozen sections were found to show an increasing amount of major transverse cracking with time; however, four times that number showed decreasing trends in major cracking. This effect may have been maintenance related, but this could not be confirmed. Similar trends in serviceability were not found; however, some sections exhibited constant or fluctuating PSI, while others showed simultaneous decreases in major transverse cracking and serviceability. Some examples are provided in Figures 32 through 39.

Less than three percent (3%) of the Washington rigid pavement data involved cracking in moderate or severe categories (distress levels 12, 13, 22, 23, 31, 32 or 33). The individual bump counts for pavements with moderate or severe levels of cracking are listed in Table 25. No correlation between cracking and roughness is evident.

Reference 60 also reported transverse cracking data for rigid pavements. For JCP and JRCP, transverse cracking was recorded in four classes, as described above, in units of feet per 1,000 sq. ft. (93 sq. m.) of pavement. Unfortunately, none of the JCP or JRCP sections investigated in Reference 60 had class 3 or 4 transverse cracking exceeding 20 ft. (6 m.) per 1,000 sq. ft. (93 sq. m.). In addition, only 2 JCP sections and 5 JRCP sections had class 3 or 4 transverse cracking exceeding 10 ft. (3 m.) per 1,000 sq. ft. (93 sq. m.). However, 22 sections of CRCP were found with at least 100 transverse cracks per 1,000 feet (305 m.) of pavement. Figure 35 shows serviceability as a function of number of transverse cracks for these CRCP pavement sections. Note that the range of PSR exhibited in this figure is somewhat limited, since none of the pavement sections were rated below PSR of 3.9. Only one CRCP section showed any longitudinal cracking, and only one JCP section was found with major (class 3 and 4) longitudinal cracking exceeding 5 ft. (1.5 m.)/1000 sq. ft. (93 sq. m.).

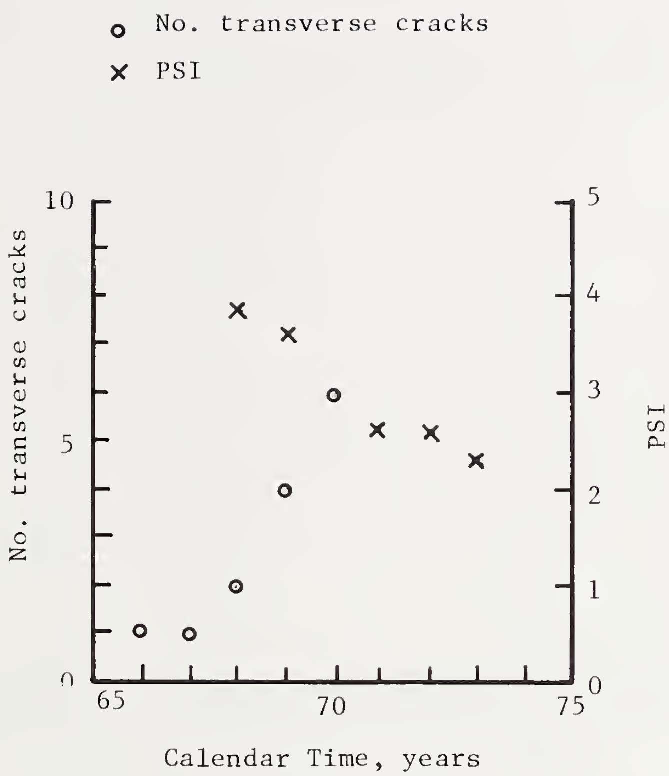


Figure 32 Serviceability and transverse cracking history, rehabilitated AASHO Test Road section 076

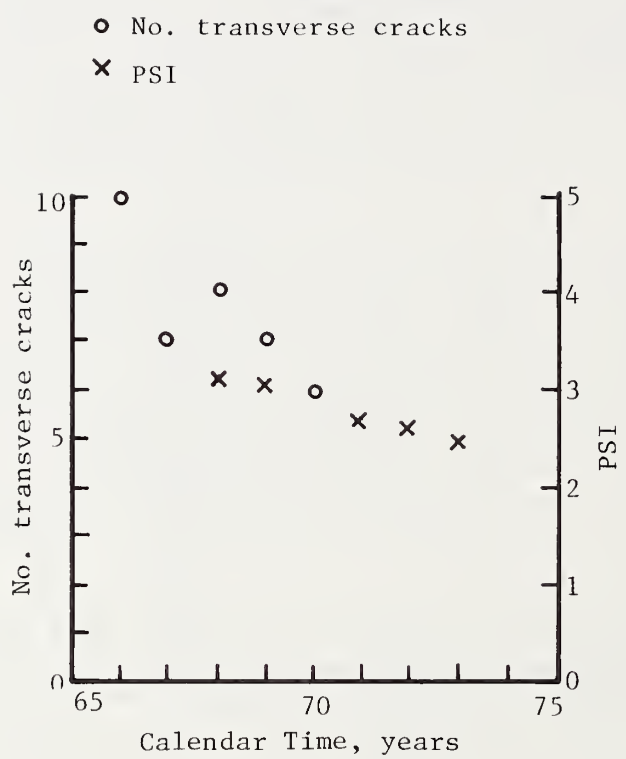


Figure 33 Serviceability and transverse cracking history, rehabilitated AASHO Test Road section 404

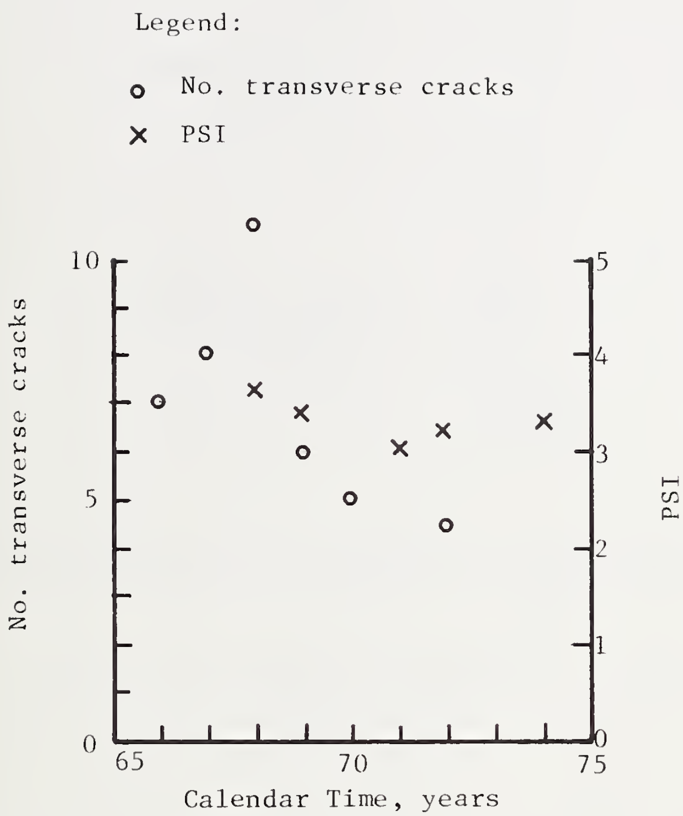


Figure 34 Serviceability and transverse cracking history, rehabilitated AASHO Test Road section 338

TABLE 25 BUMP COUNT VALUES FOR MODERATE AND SEVERE CRACKING, WASHINGTON RIGID PAVEMENTS

Distress Level		Bump
Severity	Extent	Counts
1	2	716
1	3	553, 553, 553 478, 478
2	2	416
2	3	-
3	1	817, 791, 415 675, 223, 223, 352, 340 145
3	2	866, 55, 301
3	3	-

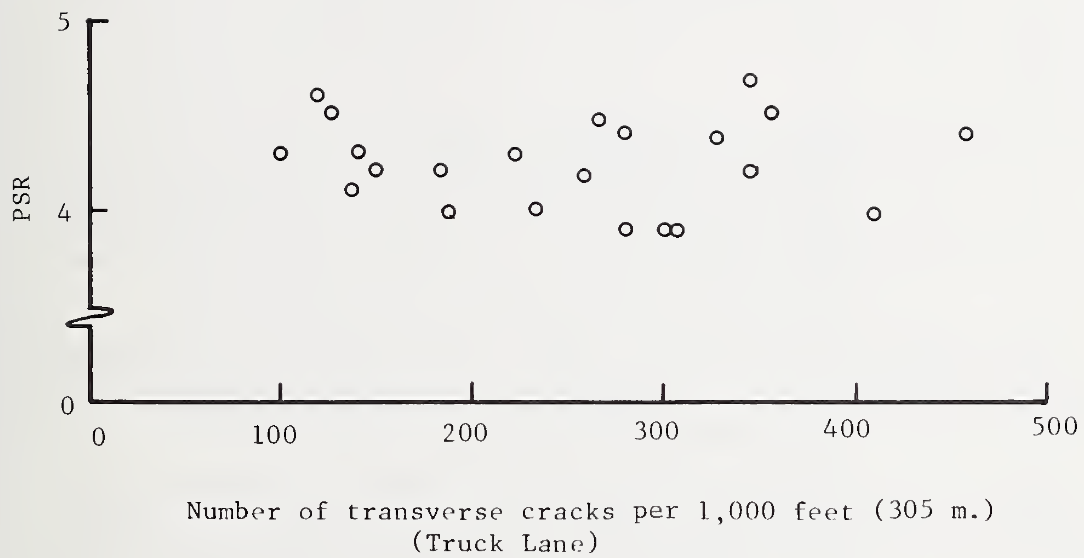


Figure 35 PSR as a function of transverse cracking, CRC pavement

Data from the original AASHO Road Test offered the best potential for modeling cracking/serviceability relationships. Based upon examination of data from Loops 3, 4, 5 and 6, the amount of major cracking was selected as the reported cracking variable most suited for such modeling. The relationship between major cracking (class 3 and 4 cracking, measured in ft. per 1000 sq. ft. (93 sq. m.)) and serviceability is illustrated in Figure 36 for pavement sections from Loop 6. Only those sections which exhibited a change in major cracking with no change in patching were included in this Figure.

The relationship between major cracking and serviceability for these pavements may be described by the equation:

$$PSI = PSI_o - 0.05\ (MCI-25),\ \text{for}\ MCI > 25 \quad . . . . (9)$$

where:

PSI = present serviceability index

PSI_o = initial serviceability index (before the occurrence of major cracking exceeding 25 ft. (8 m.) per 1,000 sq. ft. (93 sq. m.))

MCI = major cracking index = amount of class 3 and class 4 cracking, ft. per 1,000 sq. ft. (93 sq. m.)

The threshold value of 25 ft. (8 m.) per 1,000 sq. ft. (93 sq. m.) was chosen to account for the observation that no significant effect on serviceability was observed for less extensive major cracking.

A reasonable agreement between the PSI calculated from equation (9) and the observed PSI for the AASHO Road Test sections was found. Table 26 lists the observed PSI values alongside those predicted by equation (9) for several pavement sections which exhibited appreciable major cracking with little or no patching.

Both Reference 60[1] and the rehabilitated AASHO Test Road data report D-cracking for JCP and JRCP. Only one JCP section and one JRCP section from Reference 60 included D-cracking exceeding 8 per 1,000 ft. (305 m.) of pavement, but many of the rehabilitated AASHO sections had D-cracking of this extent. Figure 37 shows serviceability plotted against D-cracking for thirty-five pavement sections from the rehabilitated AASHO Test Road.

Faulting. Of the data sources described in Chapter 3, Georgia, Washington, and Illinois (Rehabilitated AASHO Test Road) include faulting data. Georgia, however, did not report serviceability or roughness, so

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[1]Nussbaum, P.J. and E.C. Lokken, "Portland Cement Concrete Pavements, Performance Related to: Design - Construction - Maintenance", Report No. FHWA-TS-78-202, August 1977.

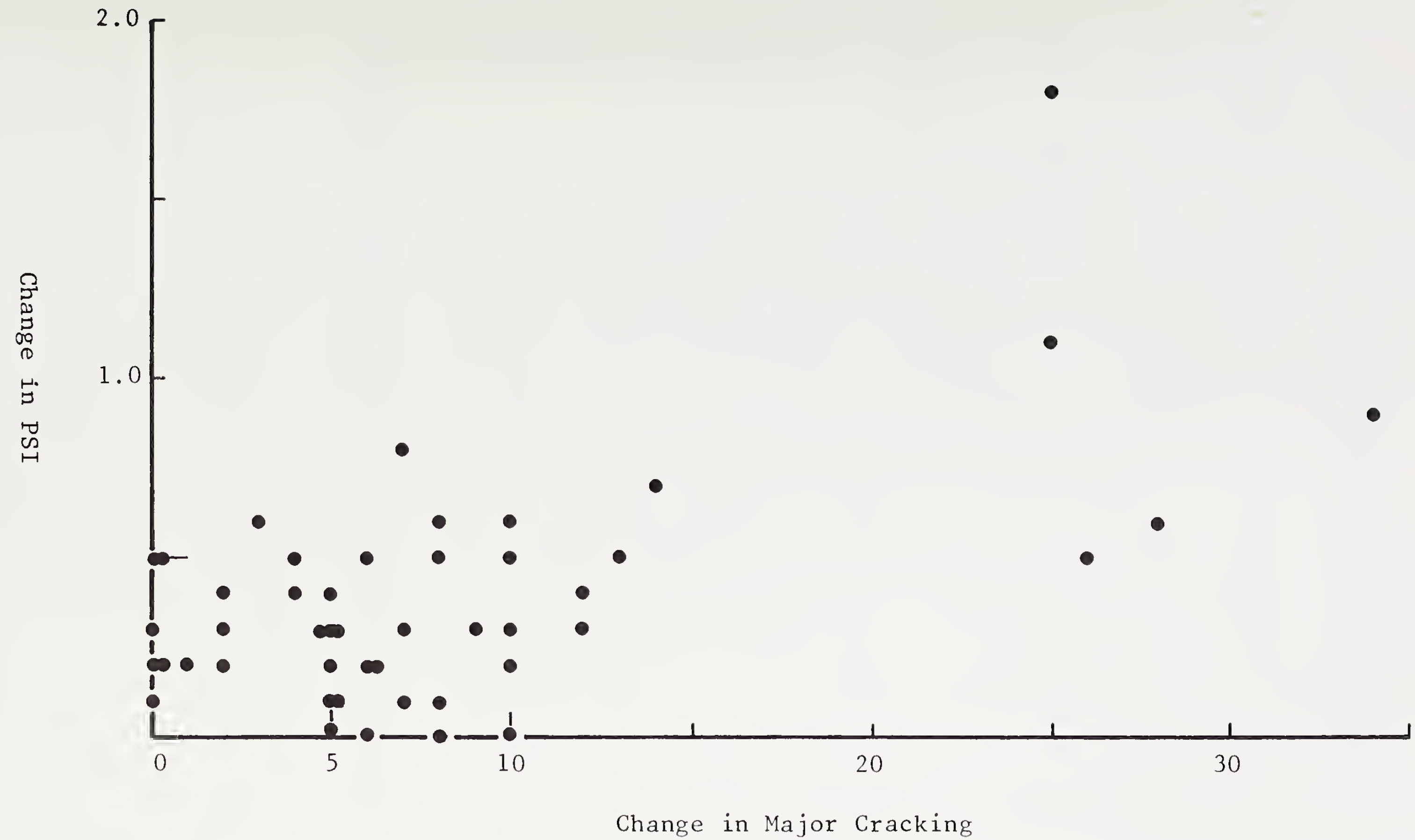


Figure 36 Change in serviceability versus change in major cracking for selected Rigid pavement section, AASHO Road Test

TABLE 26 SERVICEABILITY RELATED TO MAJOR CRACKING, AASHO ROAD TEST RIGID PAVEMENTS

Section	Observed	Predicted by equation (9).
400	3.0	3.0
	2.7	2.5
	2.2	2.2
384	3.5	3.7
	2.8	3.0
	3.0	3.0
	2.7	3.0
	2.4	2.9
	1.5	1.2
370	3.0	3.4
	3.4	3.1
	3.2	3.1
	2.6	3.0
	2.7	2.7
	2.5	2.4
	2.0	1.7
	1.5	1.5
374	2.8	3.2
	3.0	3.1
	2.5	3.1
	2.0	1.8
	1.5	1.4

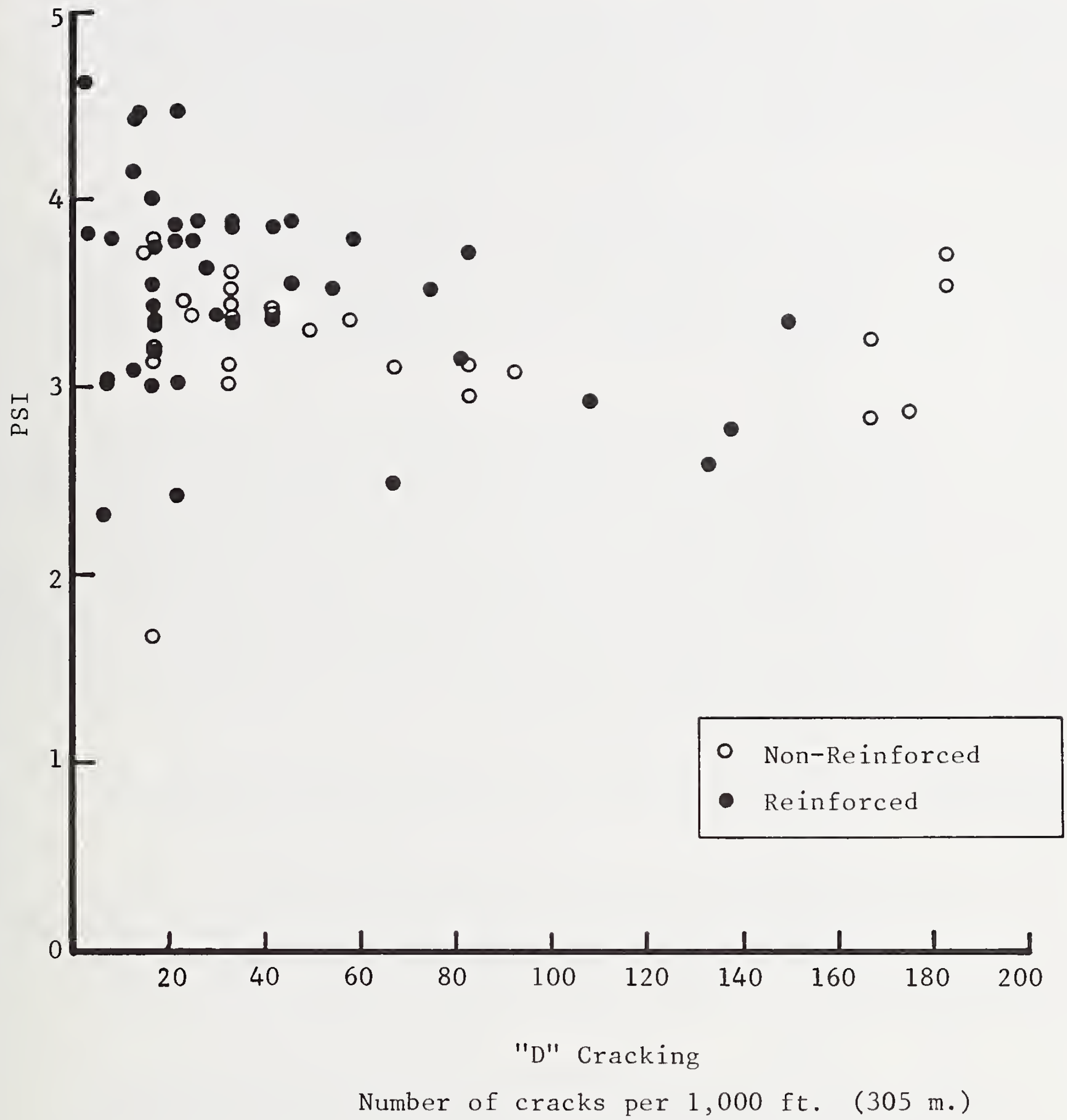


Figure 37 Serviceability related to "D" cracking, rehabilitated AASHO Test Road

that this source could not be used in determining serviceability/faulting relationships.

None of the JCP sections at the Rehabilitated AASHO Test exhibited significant faulting, but approximately one-third of the JRCP sections exhibited average joint faulting greater than 0.15 inches (0.38 cm.). Serviceability is plotted as a function of faulting for these pavements in Figure 38. No trend toward decreasing serviceability with increasing faulting is evident from these data.

Less than three percent (3%) of the Washington JCP data involved faulting of moderate or severe extent. A large number of sections did exhibit moderate (1/4" to 1/2") (0.6 to 1.2 cm.) or severe (>1/2") (>1.2 cm.) faulting over a small area (less than 16% of panels). Table 27 shows the bump counts for these sections. No relationship between faulting and bump count was found.

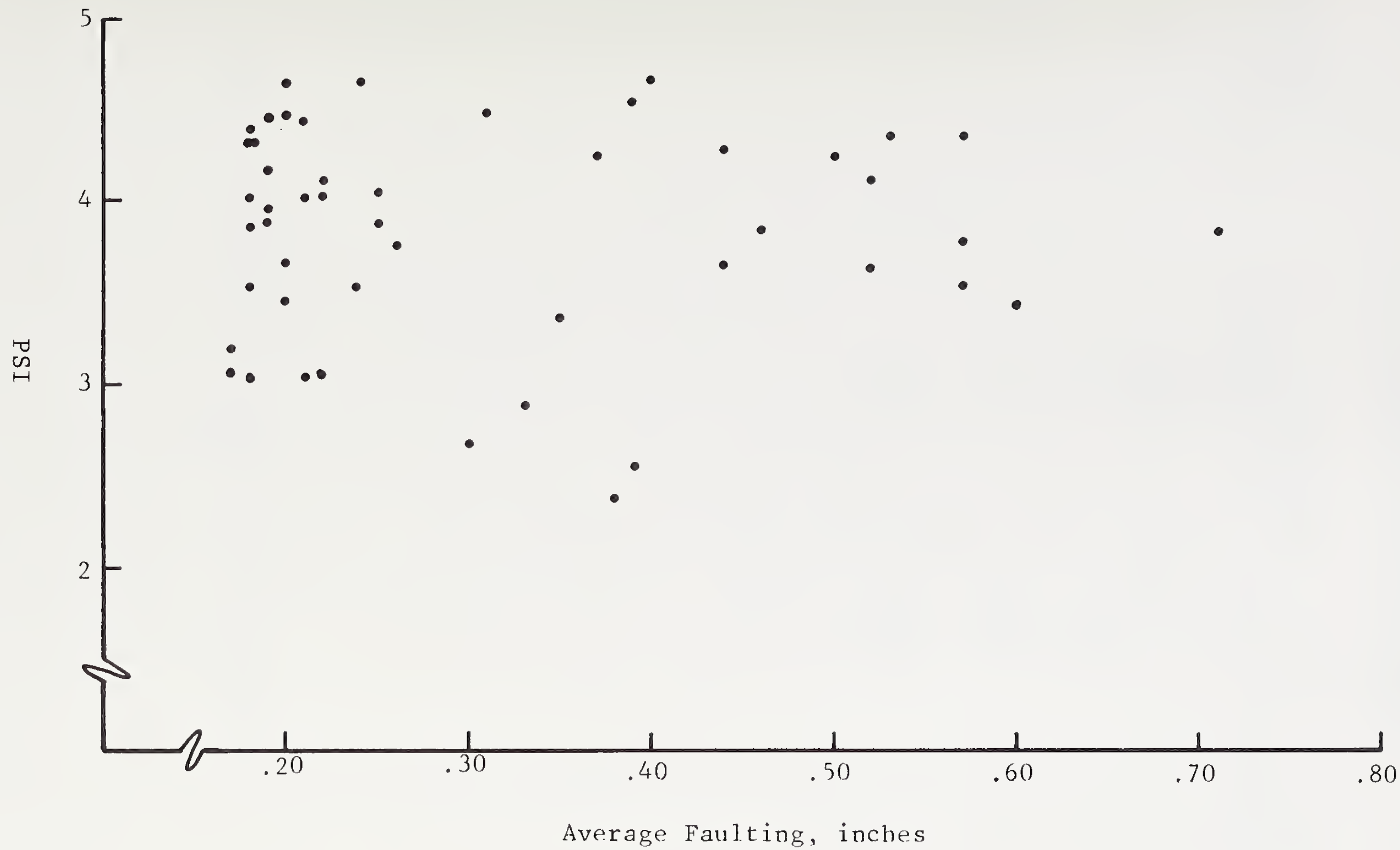
Reference 60 reports joint and crack faulting for JCP and JRCP. However, no significant crack faulting was reported in either case, and only seven pavement sections of JRCP showed any joint faulting whatsoever. Twenty-five of the JCP sections were moderately or severely faulted, though (faulting greater than 0.13 inches (0.33 cm)). Faulting for these sections is reported in three categories: less than 1/8" (0.32 cm), 1/8" to 1/4" (0.32 to 0.64 cm.) (moderate), and greater than 1/4" (0.64 cm.) (severe), with number of joints faulted per 1,000 ft. (305 m.) of pavement recorded in each case. Figure 39 illustrates PSR versus moderate and severe faulting for these JCP sections.

Spalling. The Washington data and Illinois (Rehabilitated AASHO Test Road) data include spalling, as does the data set provided in Reference 60.

Table 28 shows the mean bump count for each severity and extent level of joint spalling reported in the Washington data base. One might expect that mean bump count would increase as spalling becomes more severe and widespread. However, this is not the case for the data presented here.

Two dozen test sections from the Rehabilitated AASHO Test Road showed major spalling exceeding 8 spalls per 1,000 ft. (305 m.) of pavement. Spalling is reported in this data base as number of spalls observed in each of our classes, where class 3 + class 4 spalling is considered "major". Since the test section lengths varied, it was necessary to convert the number of spalls per section to number of spalls per 1,000 ft. in order to compare spalling between different sections.

Figure 40 illustrates the Present Serviceability Index as related to major spalling for the Rehabilitated AASHO data. From this figure it appears that there is no strong relationship between spalling and serviceability, at least in the limited range of spalling represented in the figure.



1 inch = 2.54 cm.

Figure 38 PSI versus joint faulting for pavement with average faulting exceeding 0.16 inches, rehabilitated AASHO Test Road

TABLE 27 BUMP COUNTS BY LEVEL OF FAULTING, WASHINGTON RIGID PAVEMENTS

Faulting Level		Bump Count		Number of
Severity	Extent	Mean	Std. Deviation	Measurements
	N	619.6	483.8	159
1	1	792.8	506.9	276
1	2	735.0	203.3	7
1	3	1030	0.0	1
2	1	430.6	352.6	105
2	2	791.0	0.0	1
2	3	-	-	0
3	1	811.8	383.7	84
3	2	869.7	504.6	7
3	3	-	-	0

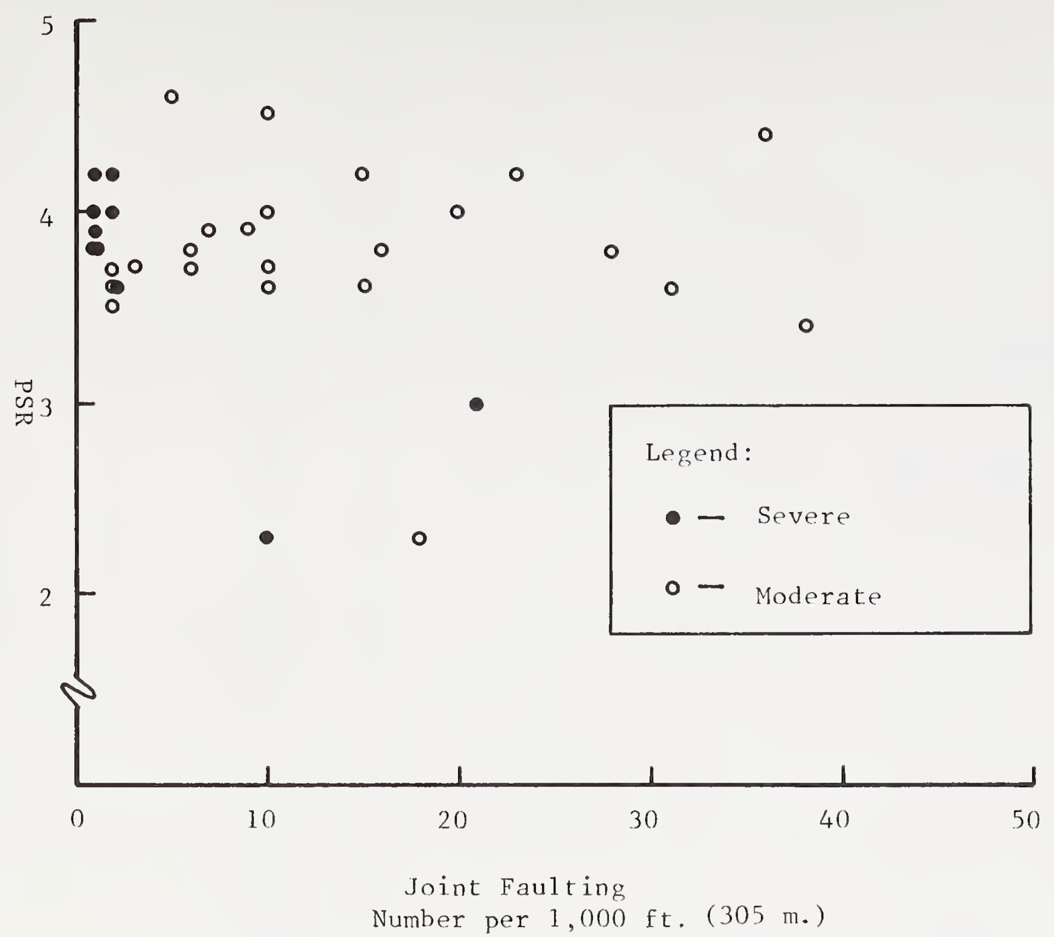


Figure 39 PSR versus joint faulting for moderately or severely faulted pavements (Ref.60)

TABLE 28 BUMP COUNTS BY LEVEL OF JOINT SPALLING
WASHINGTON RIGID PAVEMENTS

Joint Spalling		Bump Count		Number of
Severity	Extent	Mean	Standard Deviation	Measurements
	N	381.5	230.4	70
1	1	801.0	545.2	284
1	2	810.5	382.3	59
1	3	-	-	0
2	1	569.8	447.8	116
2	2	809.4	304.8	23
2	3	-	-	0
3	1	804.0	18.4	2
3	2	654.3	382.0	83
3	3	55.0*	0	1

* This value is probably an error in coding.

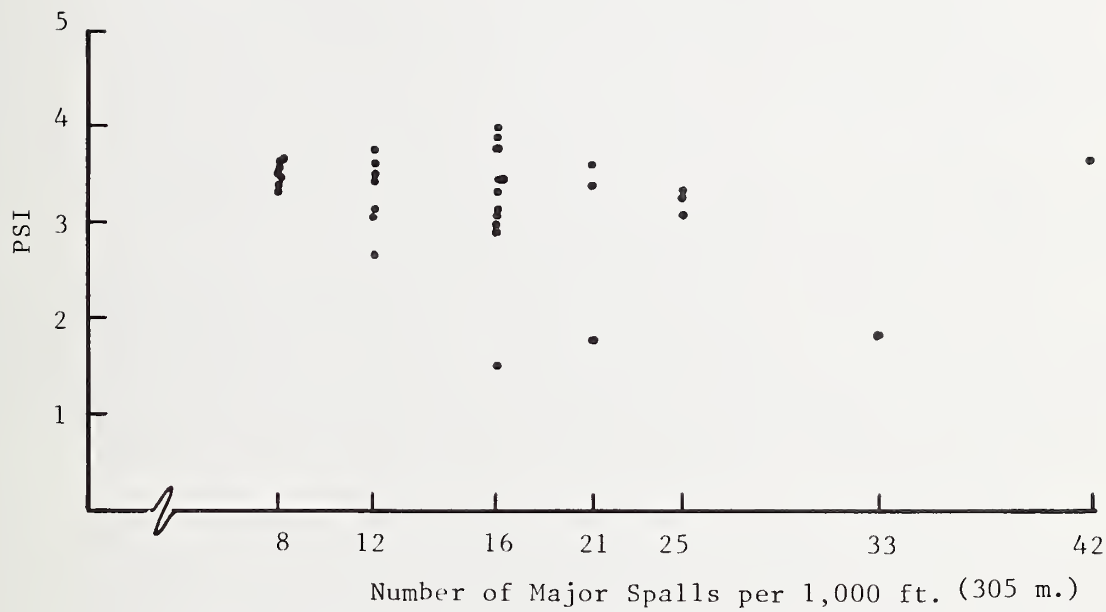


Figure 40 Serviceability versus major spalling, rehabilitated AASHO Test Road

Reference 60 reports crack spalling for continuously reinforced concrete pavements, and both joint spalling and crack spalling for plain jointed and jointed reinforced concrete pavements. In each case the number of spalls per 1,000 ft. of pavement is recorded. However, only two sections of each pavement type exhibited either crack or joint spalling exceeding 8 spalls per 1,000 ft. (305 m.) of pavement.

General Distress/Serviceability Relationships. Multiple regression analyses were performed on combined data records from forty-nine rigid pavement sections, as listed in Table 29. The data were obtained from Reference 3, and include values for PSR determined by a rating panel as well as PSI calculated from the AASHO equation:

$$PSI = 5.41 - 1.80 \log_{10} (1 + \overline{SV}) - 0.09 \sqrt{C+P} \quad . . . . . (10)$$

or, using the natural logarithm,

$$PSI = 5.41 - 0.78 \log_{e} (1 + \overline{SV}) - 0.09 \sqrt{C+P} \quad . . . . . (11)$$

This allows direct comparison of the results obtained here with those reported in Reference 3 for the AASHO equation.

Data from Table 29 were input into the computer program ALLRSQ, which computes R^2 for all possible regressions that can be formed from subsets of twenty or fewer "independent" variables. ALLRSQ was used to parsimoniously examine a full factorial of combinations of independent variables and transformations of those variables, in order to select specific combinations of variables for more thorough investigation.

Table 30 lists the variables and transformations input to program ALLRSQ. The combinations of variables found to produce a significant increase in R^2 and for which R^2 exceeded 0.90 are listed in Table 31. These most significant combinations were chosen for input to program FIXREG, along with PSR values, construct PSI equations (i.e., equations specifying PSI as an estimator of PSR, based on distress measurements). The statistics calculated by program FIXREG are reported in Table 32.

This analysis showed that the equation:

$$PSI = 5.41 - 0.73 \log_{e} (1 + \overline{SV} + 0.0063 (C+P) - 0.17 \sqrt{C+P} . . (12)$$

yields a significant (at $\alpha = 0.05$) increase in R^2 and a decrease in S^2 with a much lower C_p than equation (11). However, the plot of residuals for equation (12) above exhibits extreme departures which suggest lack-of-fit. The plots of residuals vs. observation and residuals versus Y with a little imagination exhibit a slight oscillation about the abscissa. However, this trend appears to be slight and the residuals are thought to be spread more or less evenly about the residual space. The plot of

TABLE 29 DATA FOR 49 SELECTED RIGID PAVEMENTS (REF. 3)

Pvt. Loc.	Sect. Code	Present Serviceability Ratings, AASHO Panel: 1st PSR	Replic. diff. in PSR	std. dev of PSR among raters	Acceptability Opinions, AASHO Panel, Fraction: Yes	No	Longitudinal Roughness: $\overline{SV}$ Mean Slope Var'nce in wh'pth (x10⁶)	F Fault'g in Wh'pth in in/1000'	Cracking: C Class 2 and Sealed Cracks ft/1000 ft²	Spalling: ft.²/1000ft.² for areas > 3" Dia.	Patching: P Patch'd Area ft²/1000ft.²	PSI 211: Pres. Serv. Index	Resid.: Diff. Betw'n PSR & PSI
Ill.	R1	2.0	.2	.6	.0	.8	52.0	2	53	4	8	1.7	.3
	R2	4.2		.3	1.0	.0	6.5	0	4	0	0	3.7	.5
	R3	2.6	.3	.6	.2	.5	22.2	0	42	0	11	2.3	.3
	R4	2.3	.2	.3	.0	.5	26.2	7	46	0	7	2.2	.1
	R5	1.2		.4	.0	1.0	47.8	1	102	0	28	1.4	.2
	R6	2.8	.1	.6	.2	.1	25.5	3	15	2	1	2.5	.3
	R7	4.4	.0	.3	1.0	.0	3.2	0	0	0	0	4.3	.1
	R8	1.1	.2	.4	.0	1.0	50.8	3	65	11	5	1.6	.5
	R9	0.9	.0	.3	.0	1.0	76.8	1	74	19	85	0.9	.0
Minn.	201	1.3	.1	.6	.0	1.0	43.3	1	40	60	59	1.6	.3
	202	1.8		.5	.0	1.0	24.2	0	23	4	66	2.1	.3
	203	2.1	.3	.6	.1	.9	24.7	0	47	1	41	2.1	.0
	204	4.1		.3	1.0	.0	2.4	0	4	0	0	4.3	.2
	205	3.8	.3	.4	1.0	.0	4.0	0	2	0	0	4.0	.2
	206	3.0	.0	.5	.6	.2	7.8	1	14	0	1	3.4	.4
	207	3.0		.6	.4	.2	7.5	1	22	0	0	3.3	.3
	208	2.9	.1	.6	.3	.4	9.7	0	14	0	0	3.2	.3
	209	2.5		.4	.1	.6	17.6	0	34	0	0	2.6	.1
	210	1.4		.5	.0	1.0	59.2	0	16	500	12	1.8	.4
	211	4.3		.2	1.0	0	3.0	0	0	0	0	4.3	.0
	212	4.3	.0	.4	1.0	0	4.0	0	0	0	0	4.1	.2
	213	3.7		.4	1.0	0	5.3	0	0	0	0	4.0	.3
	214	3.6	.3	.5	1.0	0	4.4	0	0	0	0	4.1	.5
	215	3.9		.4	1.0	0	5.3	0	0	0	0	4.0	.1
	216	3.9	.0	.6	1.0	0	6.3	0	0	0	0	3.8	.1
	217	1.3	.0	.4	.0	1.0	32.3	0	76	2	1	1.9	.6
	218	1.2		.4	.0	1.0	27.8	10	64	0	0	2.1	.9
	219	2.2		.6	.0	.9	25.6	4	97	0	1	2.0	.2
	220	4.4	.0	.3	1.0	.0	4.0	0	0	0	0	4.1	.3
Ind.	401	4.0		.3	1.0	0	6.6	2	0	1	0	3.8	.2
	402	3.8		.4	1.0	0	6.6	4	11	1	0	3.5	.3
	403	3.6		.6	.9	0	6.8	1	2	4	0	3.7	.1
	404	3.2		.6	.6	.2	9.8	4	1	1	2	3.4	.2
	405	2.6		.6	.3	.5	14.6	5	72	13	0	2.5	.1
	406	2.8		.6	.4	.3	10.4	5	70	10	1	2.8	.0
	407	1.8	.5	.6	.1	.8	49.4	1	41	4	29	1.6	.2
	408	1.8		.6	.1	.8	54.5	2	42	8	37	1.5	.3
	409	2.1		.6	.2	.8	36.6	1	50	7	29	1.8	.3
	410	2.2		.5	.2	.8	25.1	2	86	5	33	1.9	.3
	411	1.8		.5	.1	.8	45.4	0	40	6	65	1.5	.3
	412	2.7		.6	.4	.4	9.9	5	81	3	5	2.7	.0
	413	4.2		.4	1.0	.0	6.1	1	0	1	0	3.9	.3
	414	4.3		.4	1.0	.0	5.2	1	0	0	0	4.0	.3
	415	4.3		.4	1.0	.0	7.1	1	0	0	0	3.8	.5
	416	1.2	.3	.6	.0	.9	81.9	8	54	1	219	0.5	.7
	417	2.2	.0	.6	.1	.7	32.2	18	36	1	0	2.2	.0
	418	4.3	.1	.3	1.0	.0	4.6	1	0	0	0	4.1	.2
	419	2.8	.0	.7	.5	.3	12.6	2	5	2	13	3.0	.2
	420	2.7	.1	.4	.1	.3	17.8	2	5	7	16	2.7	.0
Sum		138.6	3.1	1 foot = 0.305 m.								138.6*	12.5
Mean		2.83	.13	1 sq. foot = 0.093 sq. m.								2.83*	.26
Sum of Squares		57.92										53.08*	4.84*

* Obtained from Unrounded Calculations

$$PSI\ 211 = 5.41 - 1.80 \log(1+\overline{SV}) - .09\sqrt{C + P}$$

TABLE 30 RIGID PAVEMENT VARIABLES EXAMINED BY PROGRAM ALLRSQ

Variable No.	Variable
1	$\overline{SV}$ - Mean slope variance
2	F - Faulting
3	C - Cracking
4	Sp - Spalling
5	P - Patching
6	$\mathrm{Log}_e\ (\overline{sv} + 1)$
7	$\overline{SV}$ * * 2
8	C * * 2
9	C + P
10	$\sqrt{c + p}$
11	F + C + S + P

TABLE 31 COMBINATIONS OF RIGID PAVEMENT VARIABLES SELECTED BY PROGRAM ALLRSQ FOR USE IN PROGRAM FIXREG

Variables						R^2
No.	Name	No.	Name	No.	Name	
6	$\log_e (1 + \overline{SV})$	9	C + P	10	C + P	.929
5	P	6	$\log_e (1 + \overline{SV})$	10	C + P	.928
6	$\log_e (1 + \overline{SV})$	7	$\overline{SV}^2$	10	C + P	.921
1	$\overline{SV}$	9	C + P	10	C + P	.921
4	Sp	6	$\log_e (1 + \overline{SV})$	10	C + P	.920
6	$\log_e (1 + \overline{SV})$	10	C + P	-	-	.917
3	C	6	$\log_e (1 + \overline{SV})$	-	-	.913

TABLE 32 STATISTICAL ANALYSIS RESULTS FOR RIGID PAVEMENT FROM PROGRAM FIXREG

Variables in PSI equation	R^2	F	S^2	P	C_p
1-6,9,10	.9325	69.08	.0988294	9	9
6 9	.9171	254.3	.104438	3	6.158
3 6	.9126	240.1	.110088	3	8.817
6,7,8	.9287	195.5	.09171017	4	1.2284
5,5,8	.9275	191.7	.09337801	4	1.9964
1,7,8	.92081	174.1	.10193	4	5.9342
4,6,8	.92043	173.5	.102413	4	6.1567

residuals on normal cummulative distribution paper exhibits an acceptible linear trend, thus the F statistics appear to be valid.

The plot of residuals for equation (11) above did not exhibit extreme departures of trends which suggest a lack of fit. The plot of residuals versus observation number exhibited a very slight curve trend. The plot of residuals versus Y is satisfactorily spread over the residual space. The plot of residuals on normal cummulative distribution paper exhibited a satisfactory linear trend to satisfy the normality assumption for statistical purposes.

Table 29 also includes replicate PSR ratings on the same section. These ratings were several days apart to insure that the pavement section had not changed. It may be helpful to state that the PSR was defined as the mean of a group of individual ratings. Thus, associated with the PSR is a variance of individual ratings about the PSR. However, the purpose of the preceeding regression analysis was to predict PSR. Thus, to compute the pure error associated with PSR, the difference between replicate PSRs rather than the variance about the PSR, is the proper quantity to consider.

The calculation of pure error has the following form for two observations:

$$S_e^2 = \frac{\sum_{i=1}^{k} 1/2(RSR_1 - PSR_2)^2}{k} = \frac{\sum_{i=1}^{k} 1/2(\Delta_i)^2}{k} \quad \ldots\ldots\ldots\ldots \quad (13)$$

where $\Delta = PSR_1 - PSR_2$.

From the data presented in Table 29 the pure error is:

$$S_e^2 = \frac{.435}{23} = .0189$$

The analysis of variance breakdown for lack of fit is given in Table 33. Both the AASHO equation and the equation selected by this regression analysis show significant lack of fit, although equation (12) does provide somewhat improved statistics as compared to the equation (11).

TIME DEPENDENT DISTRESS/SERVICEABILITY RELATIONSHIPS

One of the goals of this project is to develop distress/performance relationships which can be used in pavement management. For management purposes, it is necessary to make forecasts of pavement performance under expected levels of traffic, environment, maintenance, etc. Hence, equations relating current and past distress to future performance must

TABLE 33 ANALYSIS OF VARIANCE FOR LACK OF FIT, EQUATIONS 11 and 12

$$PSI = 5.41 - 0.73 \log e\,(1 + \overline{SV}) + 0.0063\,(C + P) - 0.17\sqrt{C + P} \qquad (12)$$

Source	Degrees of Freedom	Sum of Square	Mean Square	F-Ratio
Residual	45	4.1269575	.09171017	
Pure Error	23	.435	.01891	
Lack of Fit	22	3.69196	.16781625	8.873 significant at $\alpha = .01$

$$PSI = 5.41 - 0.78 \log e\,(1 + \overline{SV}) - 0.09\sqrt{C + P} \qquad (11)$$

Source	Degrees of Freedom	Sum of Square	Mean Square	F-Ratio
Residual	46	4.8041467	.1044397	
Pure Error	23	.435	.01891	
Lack of Fit	23	4.3691467	.1899629	10.04 significant at $\alpha = .01$

be sought. The development of such equations is discussed in this section.

Flexible Pavements

The development of serviceability and distress with time follows certain primary patterns, as discussed previously. However, actual numerical values were found to vary considerably among pavement sections, with little apparent functional relationship to time or accumulated load.

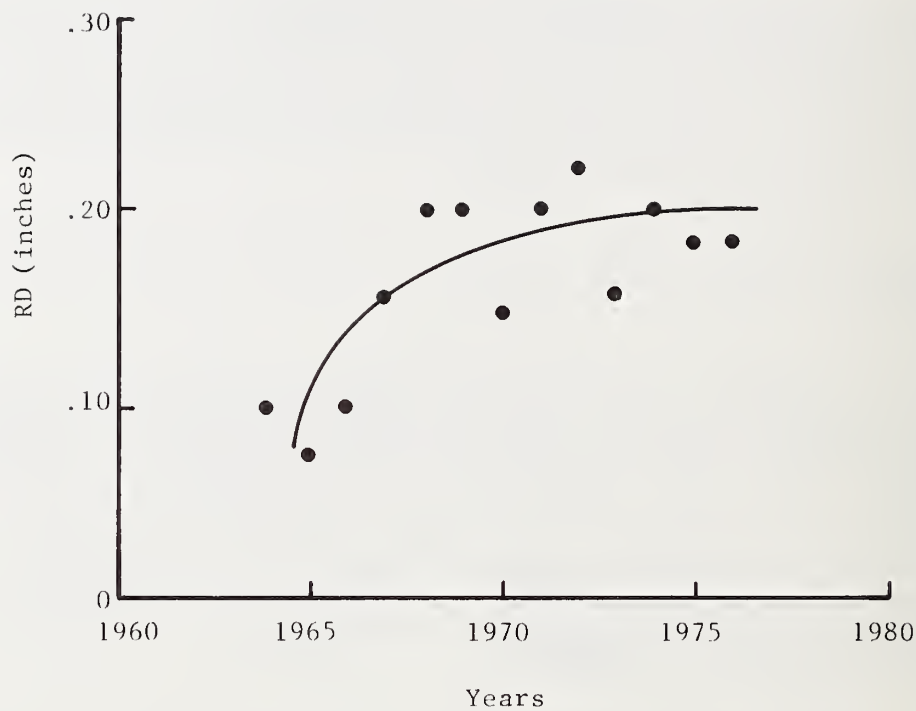
Figures 41 and 42 show the primary rutting and cracking plus patching patterns observed in the Minnesota data. Approximately half of the fifty test sections followed the general rutting pattern of Figure 41, and about forty percent followed the general C+P pattern of Figure 42. However, a wide degree of individual variation was observed and only one in five sections exhibited distress patterns which were consistent with both Figures 41 and 42.

Approximately forty percent of the Minnesota sections followed PSI patterns similar to that of Figure 43. Again, however, considerable individual variation was noted.

No consistent relationship was found between rutting pattern and serviceability pattern, or between cracking plus patching patterns and serviceability pattern. Many different combinations of RD and C+P development were found to lead to similar PSI patterns. It is remarkable, though, that of the eleven pavement sections which were consistent with the distress patterns of both Figures 41 and 42, nine sections were also consistent with the PSI pattern of Figure 43. The remaining two sections exhibited nearly constant PSI over time. Two of these eleven sections had Gravel Equivalents less than 18, five had G.E. between 18 and 25, and four had G.E. exceeding 25.

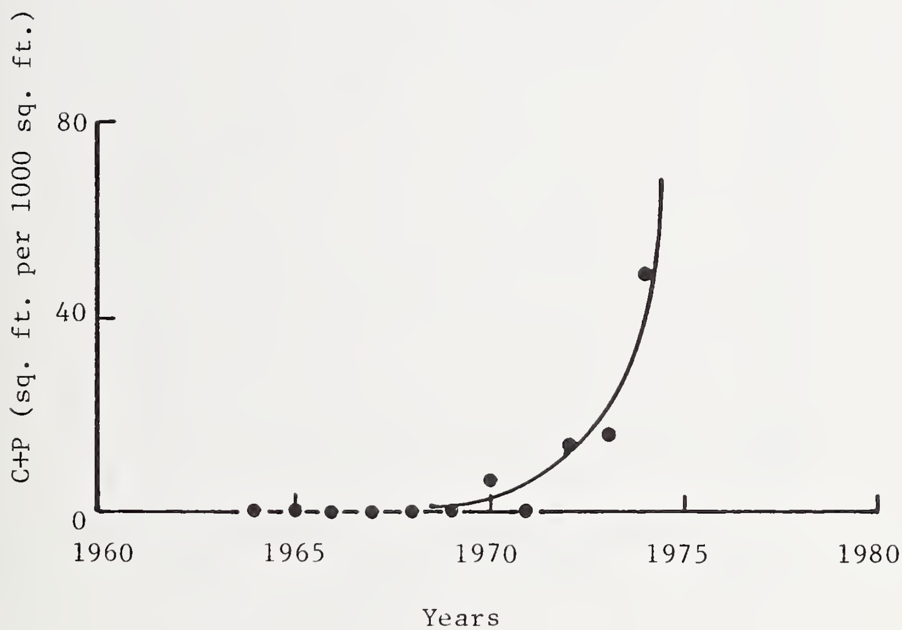
In order to incorporate other distress types into the analysis, the Texas data were examined. Figures 44 through 46 illustrate the distress and serviceability patterns observed for these data. Serviceability Index (SI), the coefficient of variation of SI (CV), and the Pavement Rating Score (PRS), are plotted in these figures. Values for the severity and extent of individual distress types are also listed.

A slight upward trend in serviceability with time was obtained as illustrated in Figure 44. This trend was observed for the majority of the pavement sections. Normally, the serviceability index should decrease with time. The general increasing trend in serviceability may be due to random errors or some type of systematic bias. The data span only a relatively short time, five years, and the increasing trend may only be a result of random noise. A bias may be built into the method of calculation of the serviceability index from Mays-ride-meters or the calibration of these instruments. Another factor which may influence these results



1 inch = 2.54 cm.

Figure 41   Rutting as a function of time, Minnesota section 14



1 sq. ft. = 0.093 sq. m.

Figure 42 Cracking plus patching (C+P) as a function of time, Minnesota Section 31

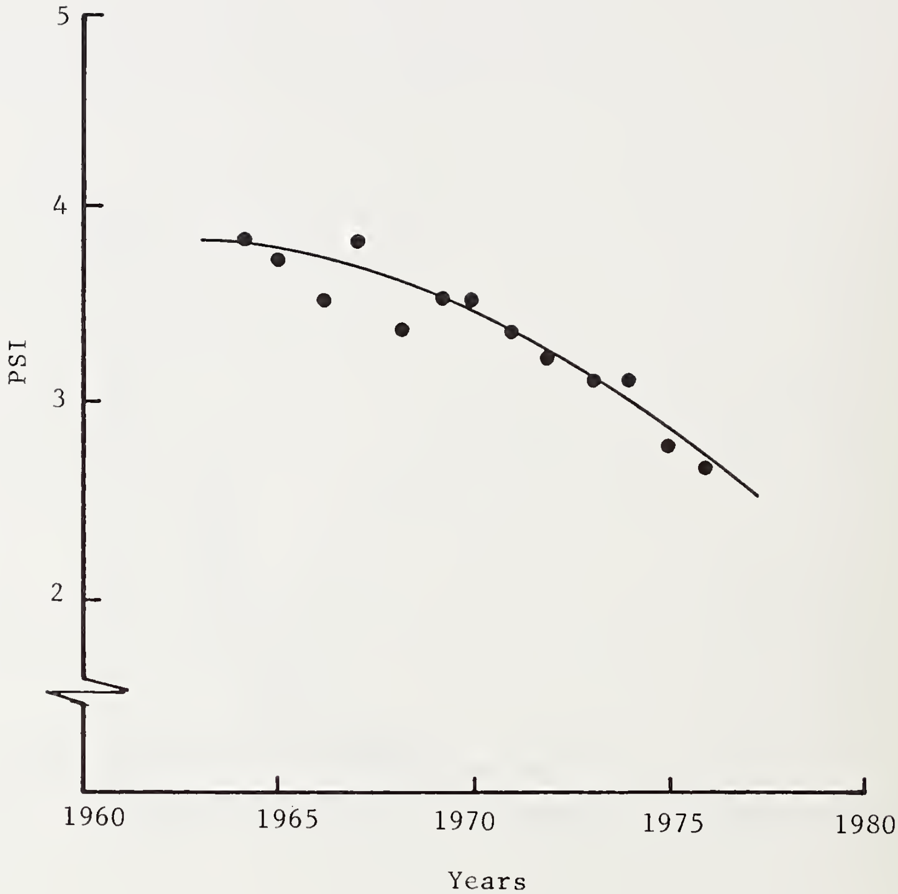
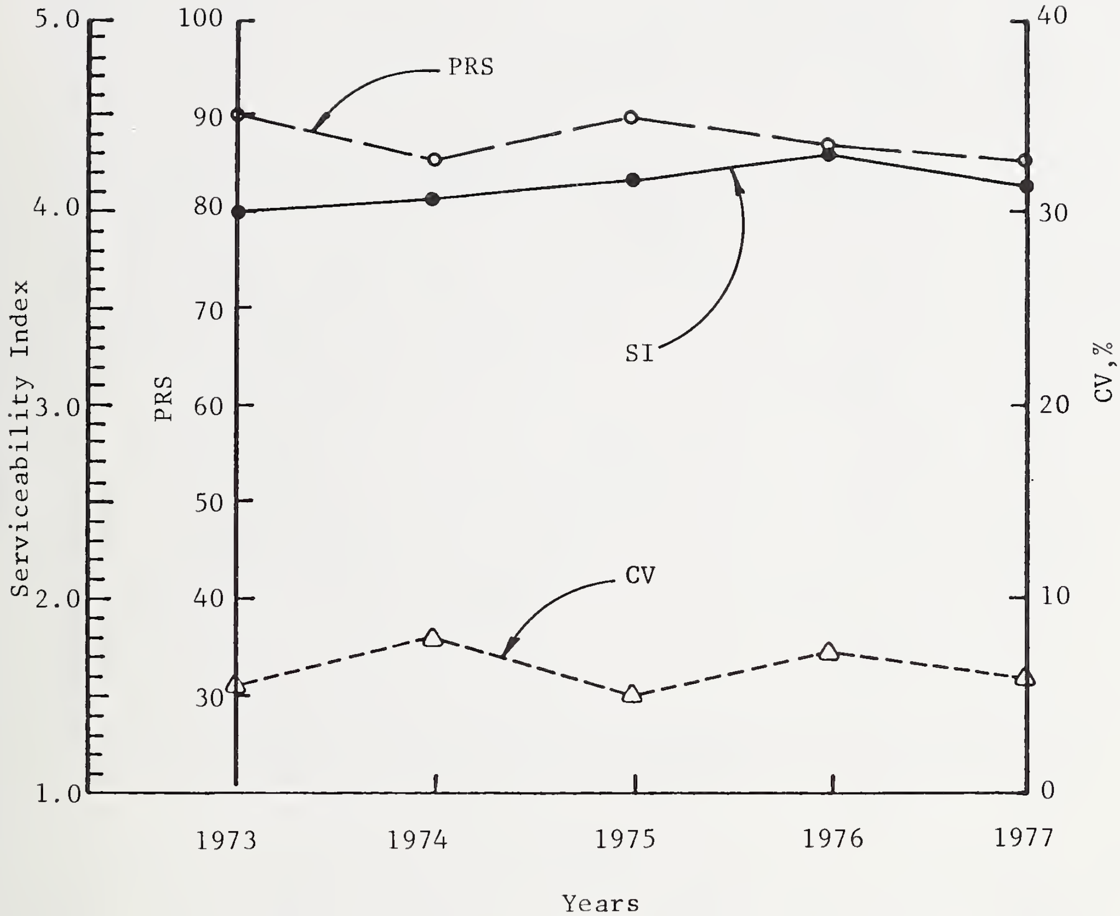
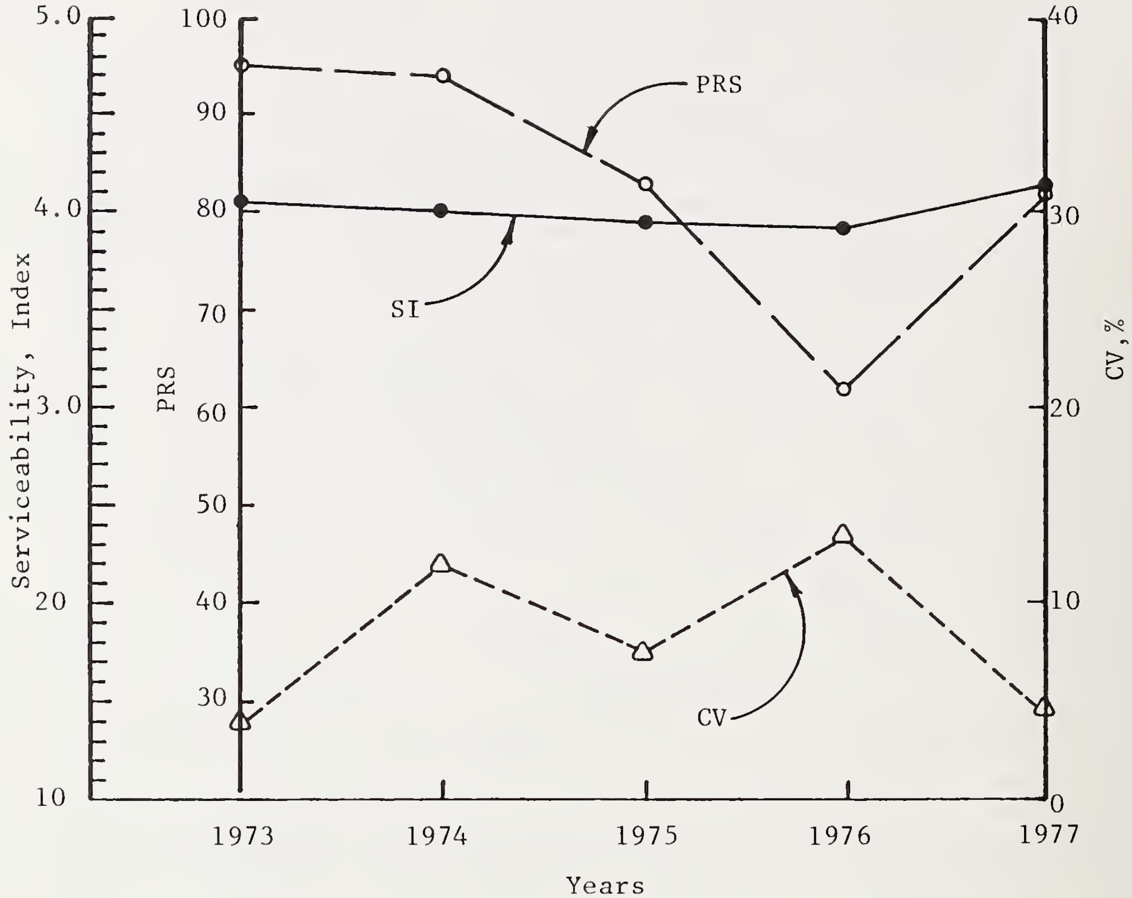


Figure 43 PSI as a function of time, Minnesota section 6



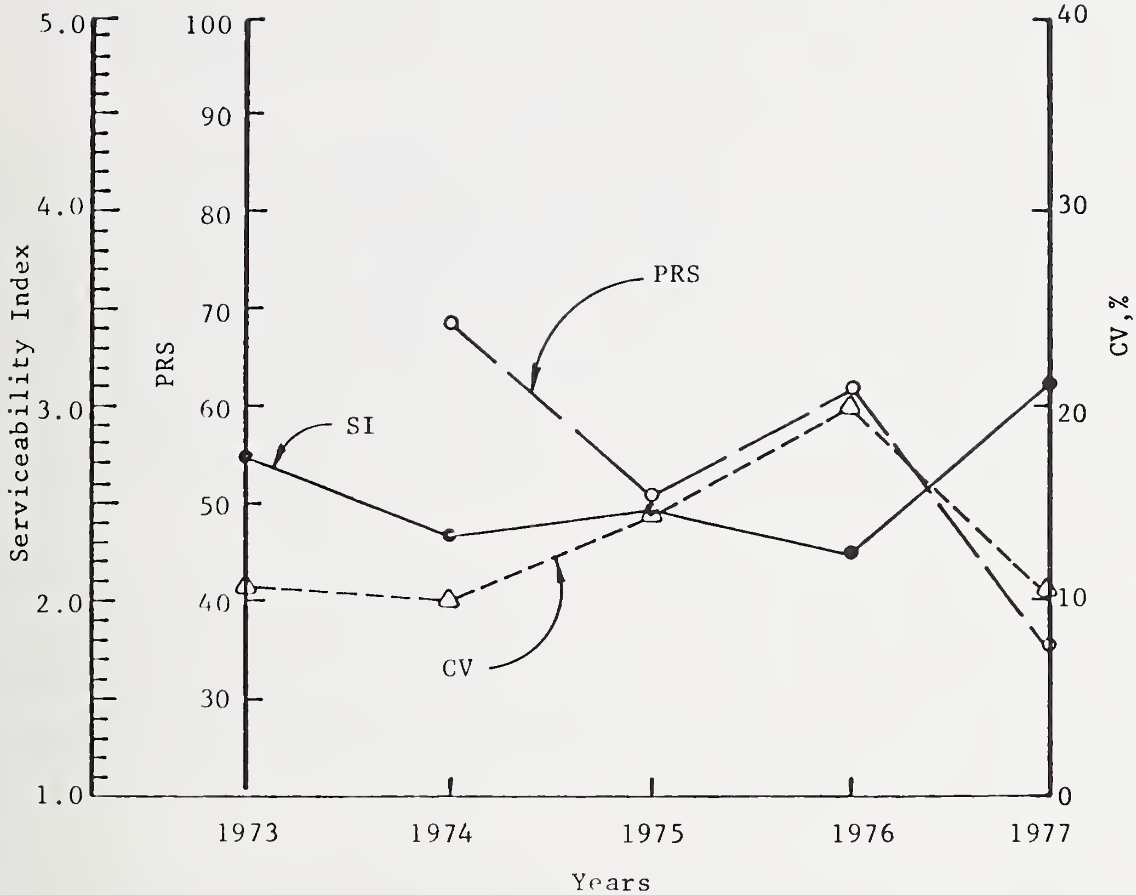
	1973	1974	1975	1976	1977
Rutting		2SL		1SL	2SL
Flush	1SL		1SL	1SL	1SL
Corr					
Ravel	1SL	2SL	1SL	2SL	2SL
Allg. Cr.					
Long. Cr.					
Trans. Cr.					
Cracks					
Patching		1F			

Figure 44 Distress and serviceability history, Texas section 704



Rutting		1SL		2SL	3SL
Flush	1SL	1SL	1MO	2SE	3SL
Corr					
Ravel					
Allg. Cr.				1SL	
Long. Cr.				1MO	
Trans. Cr.			2SL	1MO	1MO
Cracks			NS	PS	NS
Patching		1G	1G	1G	

Figure 45 Distress and serviceability history, Texas section 220



	1973	1974	1975	1976	1977
Rutting	(Not Performed)	2SL	1SL	1SL	2SL
Flush		1SL	2SL	1MO	
Corr					
Ravel					1MO
Allg. Cr.		3MO	2MO	1MO	2MO
Long. Cr.			1MO	1SL	1MO
Trans. Cr.			1MO	1SL	1MO
Cracks			NS	NS	NS
Patching		3G	3F	3F	3P

Figure 46 Distress and serviceability history, Texas section 1030

is minor maintenance performed on the various sections. This type of maintenance is not shown on the data sheets, although major maintenance is ostensibly included. However, examination of the pavement condition rating survey indicates distress occurrences such as ravelling and patching, rated one year and not rated the next.

The coefficient of variation of SI was subsequently investigated as a measure of performance. A general relation between the level of serviceability index and coefficient of variation was observed. Comparison of Figures 44 and 46 shows a relatively high coefficient of variation for the lower serviceability index in Figure 46 and a relatively low coefficient of variation for the high serviceability index in Figure 44. This inverse relation between the level of serviceability and coefficient of variation of serviceability appears logical. A pavement section with a low serviceability is more likely to exhibit greater fluctuations along its length than a section with a high serviceability. However, this general relation is not strong and does not appear to be useful as a measure of performance.

The pavement rating score was also examined as a potential measure of pavement performance. The PRS is a composite measure of pavement distress, constructed by deducting specified amounts for the level of each observed distress from a "perfect" score of 100. Thus, if PRS were adopted as a measure of pavement performance, it's relationship to pavement distress would be specified by the selection of deduct values. Unfortunately, a considerable amount of fluctuation in PRS was observed, as is evident in the Figures.

The AASHO Road Test data were used to investigate the time delay or difference in accumulated axle load between the onset of cracking and the "failure" of the pavement (the serviceability drops to the terminal level). At the AASHO Road Test, this terminal PSI level was chosen to be 1.5. Only a very weak relationship was found between the onset of cracking and the achievement of terminal serviceability, as shown in Figure 47. It is interesting, however, that the Loop 4, 5, and 6 flexible pavements used in plotting this Figure tended to experience initial cracking at distinct intervals along the accumulated load scale. The first such heavy cracking period occurred between 80,000 - 120,000 accumulated equivalent 18-kip (80 kN) loads, corresponding to April and May of 1959. The second period occurred during October and November of 1959, for loads of 270,000 - 360,000. The third period occurred during April of 1960, with corresponding accumulated load of 570,000 - 660,000. These periods are strongly correlated with high rainfall, rising or falling water tables, and spring or fall temperature fluctuations.

Notice that there are no similar trends for pavements reaching terminal serviceability. That is, pavements reached terminal serviceability individually or in small groups with some pavements "failing" in almost every month.

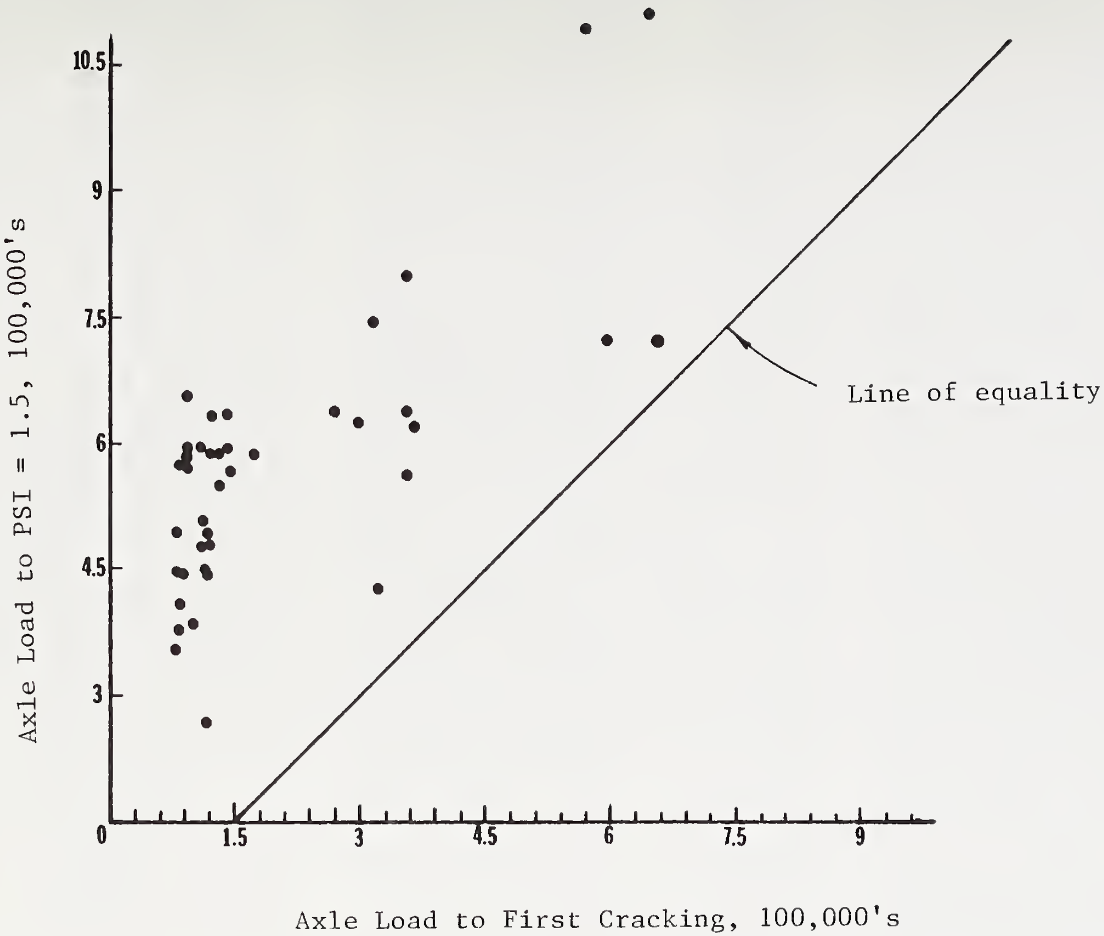


Figure 47 Delay between onset of cracking and terminal serviceability, AASHO Road Test loops 4, 5, 6 flexible pavement.

Similar results were found for the onset of major cracking, as shown in Figure 48.

Rigid Pavements

A similar analysis of AASHO Road Test data for rigid pavement sections is illustrated in Figure 49. In this case, though, the pavements experience first cracking over much larger time intervals, while pavements reach terminal serviceability in more noticeable groupings. This is also evident in Figure 50, which shows the relationship between first cracking and the pavement reaching a serviceability of 3.0. In both cases, the pavement sections grouped in the lower left-hand corner were those with slab thickness 3.5 inches (8.9 cm.), while those at the top of the figures had slab thickness 5 inches (13 cm.) or greater. A similar relationship was found for the delay between the onset of major cracking and the achievement of terminal serviceability.

Figure 51 shows the time delay between the occurrence of Damage Index (DI) values exceeding 100 ft. (30 m.) per 1000 sq. ft. (93 sq. m.) and the attainment of terminal PSI. Damage Index is a measure of total cracking and patching. It is clear that all sections failed within about 9 weeks after DI first exceeded 100.

The influence of time and accumulated axle loads on pavement serviceability was investigated, but no clear relationships were established. A clear influence of environment on serviceability for rigid pavements was found, however. This is illustrated in Figure 52. Contrary to expectation, the largest environmental influence on PSI appears to be the depth of the water table, rather than rainfall or freeze/thaw. Of course, pumping was quite a problem at the Road Test, and even a small amount of rain can be significant when the ground is already wet (high water table). However, as shown in Figure 52, the largest PSI changes occurred when the water table was low.

As a supplement to the rigid pavement data bases described previously, records of serviceability, accumulated load, and failures per mile were obtained for 125 sections of CRCP from Texas. The data were graciously made available by Dr. B.F. McCullough and his colleages at the University of Texas. The "Riding Quality" scores on a zero-to-five scale are derived from roughness data through a correlation to profilometer measurements. The number of failures per mile was obtained from condition survey records for 1974 and 1978.

Figures 53 and 54 show Riding Quality as a function of accumulated 18-kip (80 kN) equivalent axle loads and failures per mile, respectively. It is apparent that Riding Quality is independent of accumulated load and failures per mile for these CRC pavements. This may be due to maintenance activities carried out by SDHPT personnel, but maintenance records were not included in the data base.

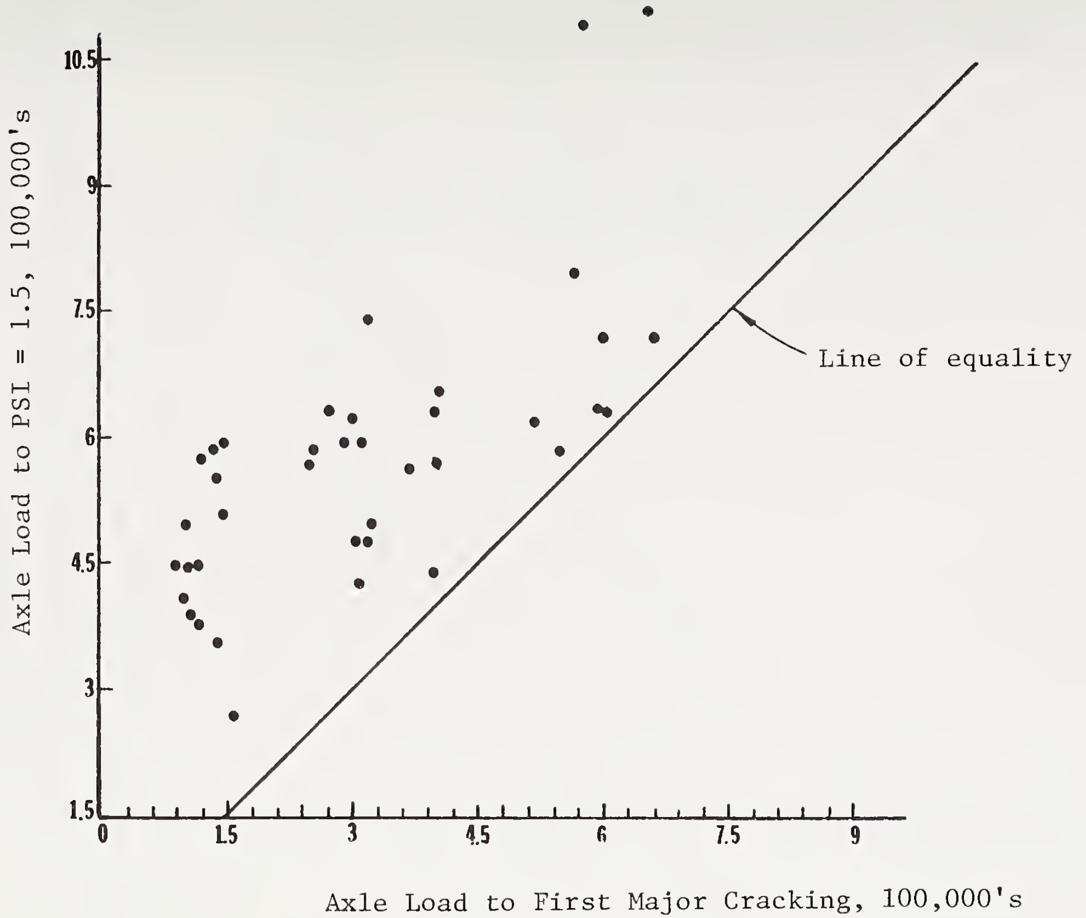


Figure 48 Delay between onset of major cracking and terminal serviceability, AASHO Road Test loops 4, 5, 6, flexible pavement.

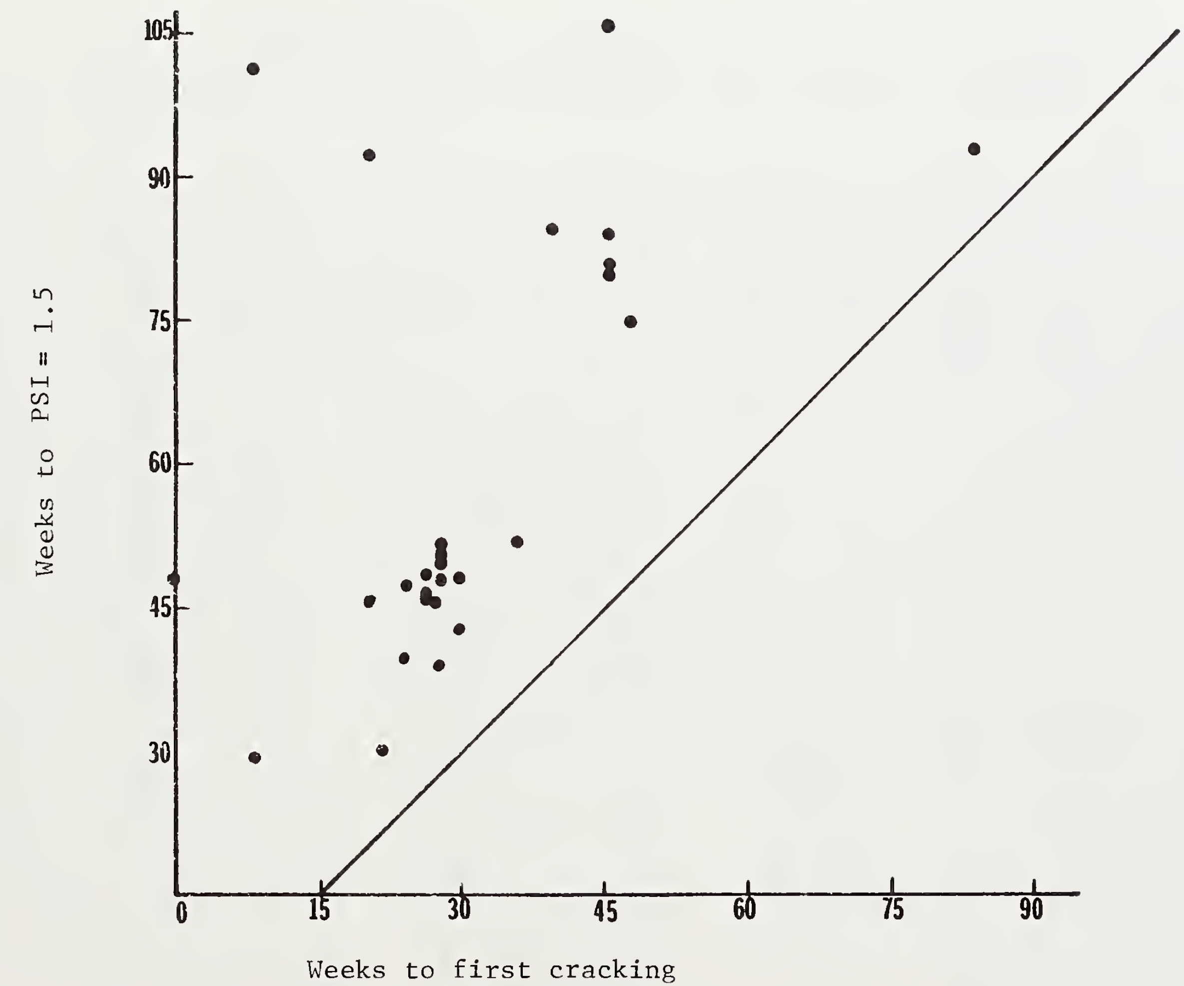


FIGURE 49 Delay Between Onset of Cracking and Terminal Serviceability, AASHO Road Test Loop 3 Rigid Pavements

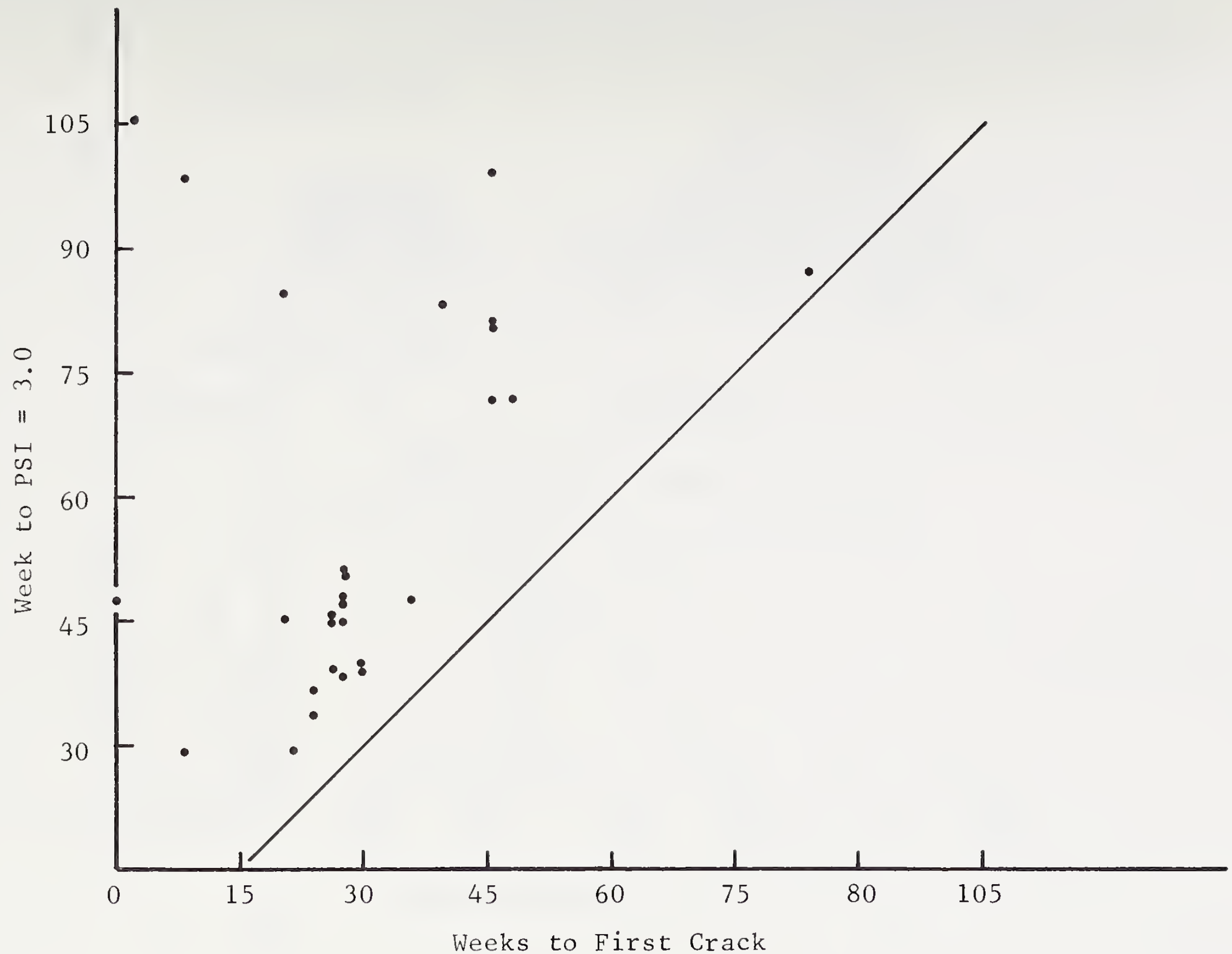


Figure 50 Delay between onset of cracking and a serviceability of 3.0, AASHO Road Test loop 3 rigid pavement

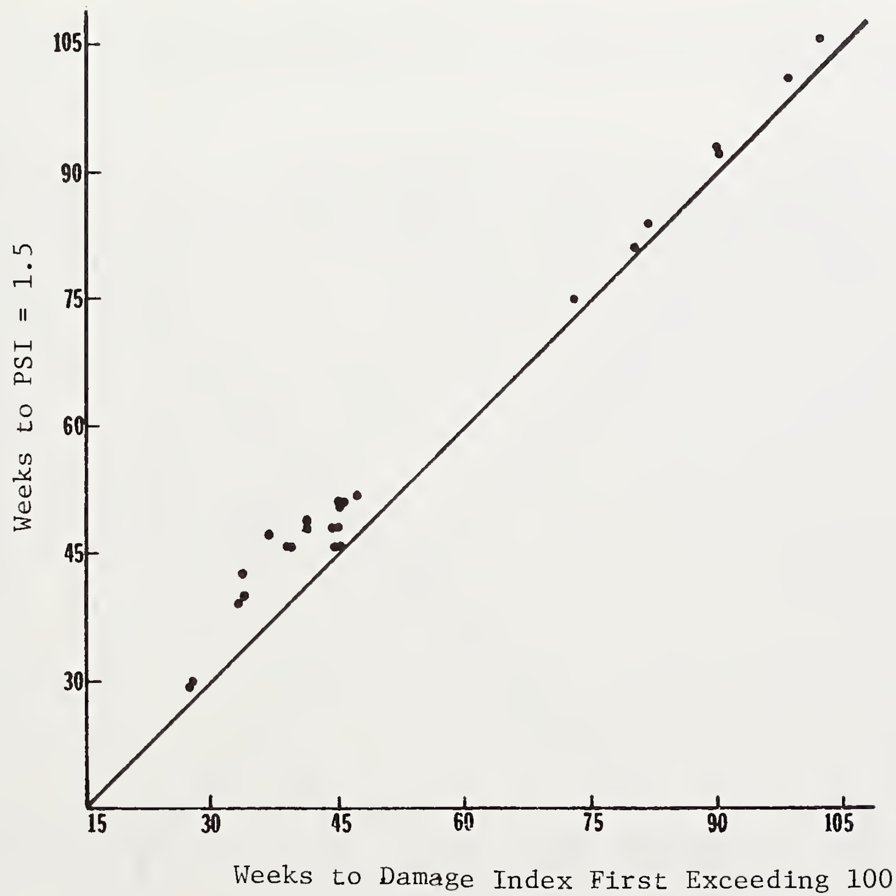
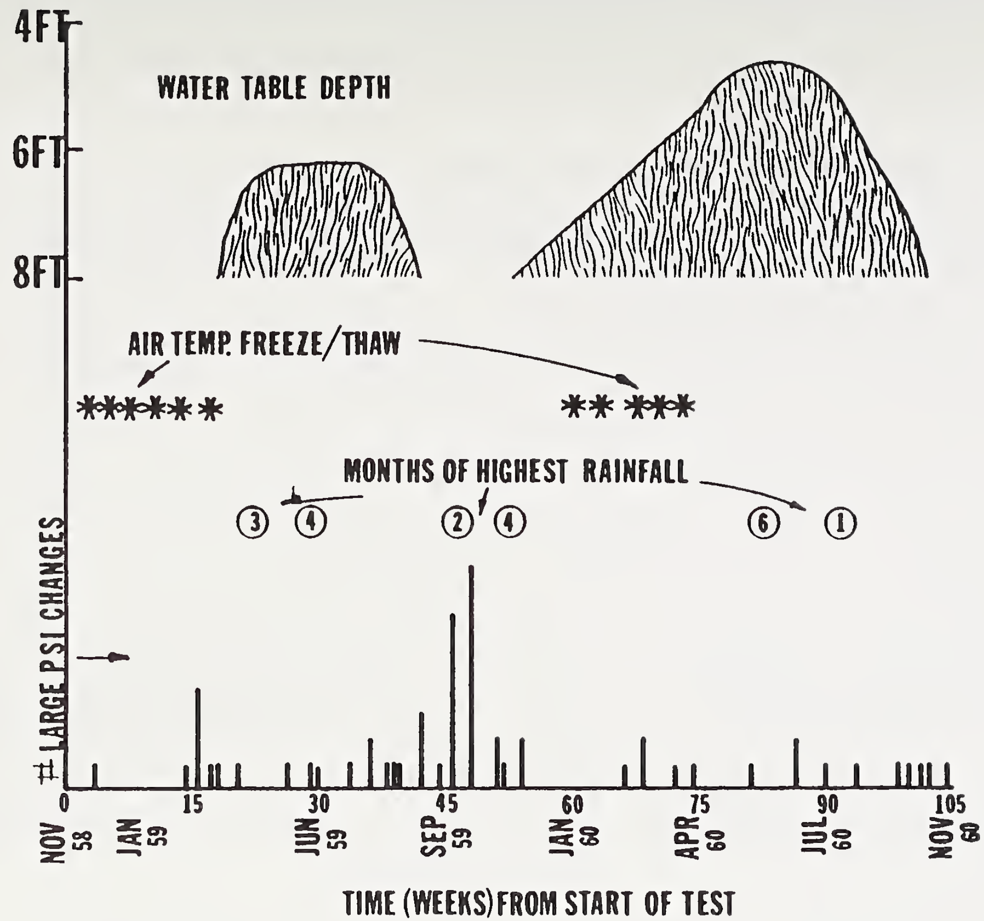


Figure 51 Delay between damage index exceeding 100 and terminal serviceability, AASHO Road Test loop 3, rigid pavement.



1 ft. = 0.305 m.

Figure 52 Influence of environmental factors on rigid pavement serviceability, AASHO Road Test

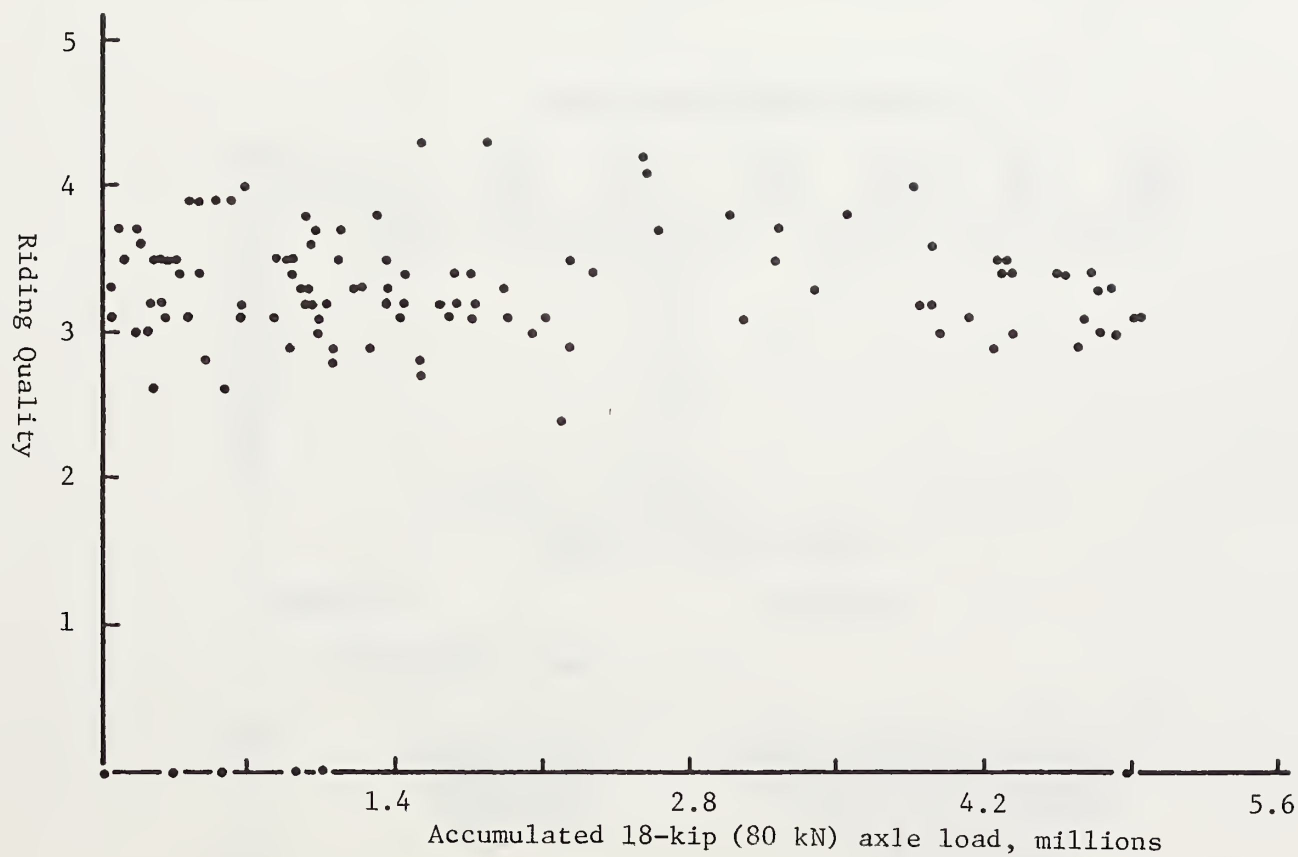


Figure 53 Riding quality versus accumulated load for several Texas CRCP sections.

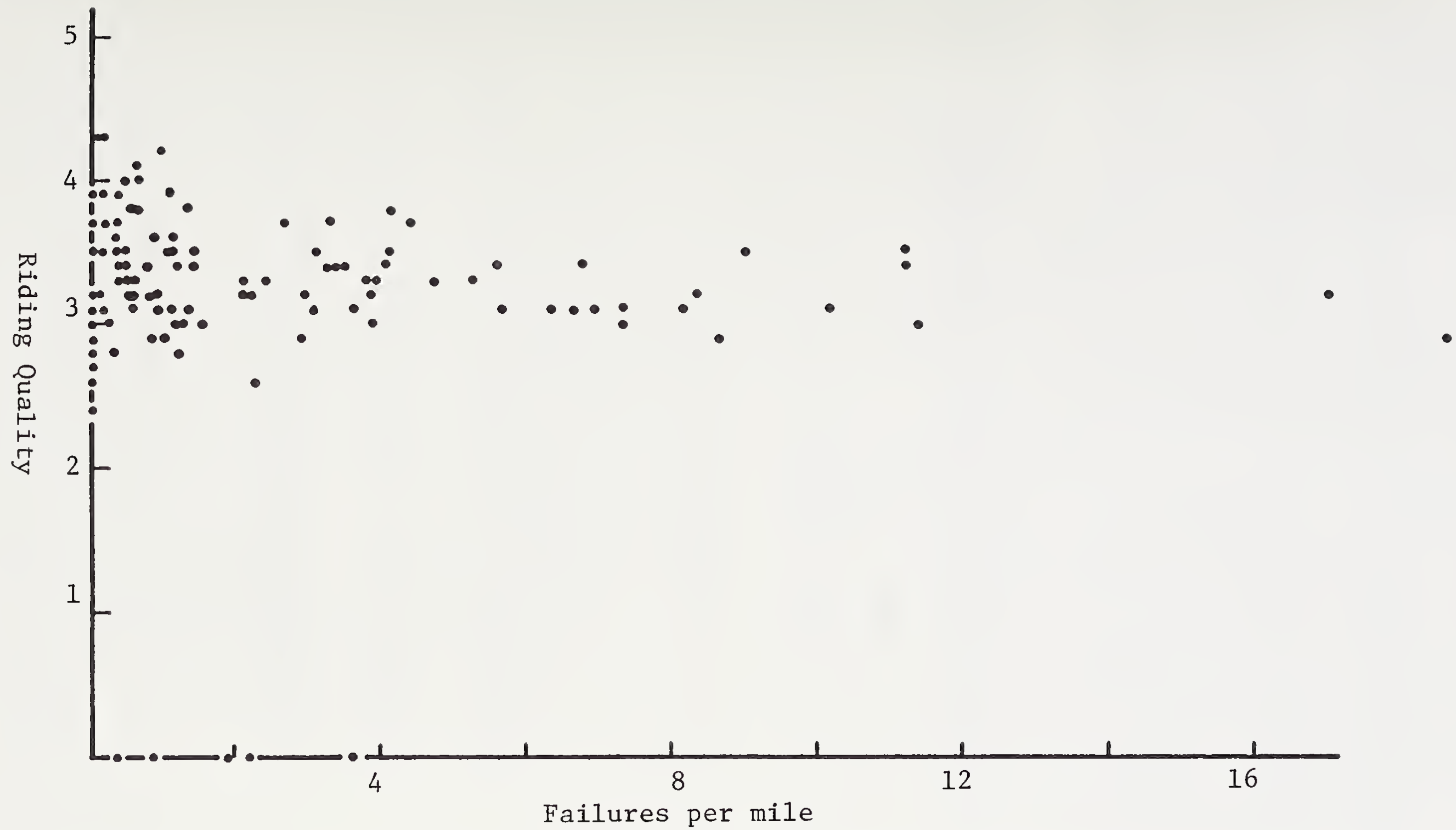


Figure 54 Riding quality versus failures per mile for several Texas CRCP sections.

1 mile = 1.6 km.

An additional modeling effort was carried out based on subjective data collected to supplement the data sources described in Chapter 3. Project staff and members of the Advisory Panels were asked to draw typical serviceability and distress histories for those pavement types, distress types, and environmental and traffic conditions with which they were familiar. These graphs were then analyzed as "average" distress and performance patterns for the stated conditions. Graphs drawn by different experts were compared, and results were combined if assumed conditions and predicted performance were sufficiently similar.

A fairly consistent set of rigid pavement subjective data was obtained for plain jointed concrete pavements, with dowels at the joints, 10-11 inch (25-28 cm.) slab thickness, granular subbase, in a Southern U.S. Climate, with routine maintenance performed at or above three-fourths the "desirable" level. Major cracking, crack spalling and joint spalling were reported as the important distress variables. Traffic was assumed to be roughly constant with time, yielding 2-10 million equivalent 18-kip (80 kN) axle loads over a 25-30 year period.

Model equations were developed from these data, expressing PSI at a given time as a function of previous distress. The individual distress equations which reproduced the subjective data most adequately are:

$$PSI\ (t) = 4.3 - 0.0097\ [C(t-7)] \quad (14)$$

$$PSI\ (t) = 4.3 - 0.061\ [Sp(t-2)] \quad (15)$$

and

$$PSI\ (t) = 4.3 - 0.25\ [JSp(t-1) - 6] \quad (16)$$

where:

PSI(t) = Present Serviceability Index at time t,

C(t) = Major (Class 3+4) cracking at time t, ft. per 1000 sq. ft.,

Sp(t) = Crack spalling at time t, percent of cracks spalled,

JSp(t) = Joint Spalling at time t, average percent of joint length spalled, and

t = Time, in years

A single equation involving all of these distress types was also developed:

$$PSI(t) = 4.3 - 0.0029\ [C(t-7)] - 0.013\ [Sp(t-2] - 0.15\ [P(t-4] - 0.045\ [JSp(t-1) -6] \quad (17)$$

where:

PSI, C, Sp, JSp, and t are as defined above, and P(t) = patching at time t, percent of area patched.

The PSI values predicted by equations 14 through 17 were compared to actual field measurements from the Rehabilitated AASHO Test Road. Figure 55 illustrates the comparison for a specific JCP section. None of the "predicted" values of PSI closely resembled the observed values for any of the two dozen JCP sections in the data base.

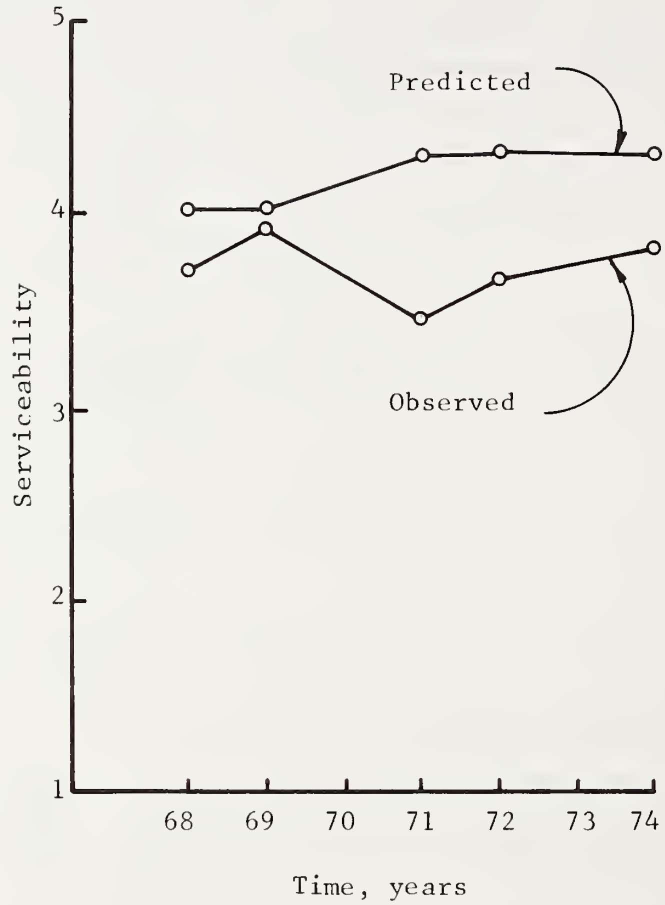


Figure 55 Comparison of predicted (equation 17) and observed serviceability histories, rehabilitated AASHO Test Road section 350

CHAPTER 6. RELATED FINDINGS

In addition to the model equations described in the previous chapter, several related findings were produced by this study. These findings are spin-offs from the modeling effort discussed previously. It is hoped that they may be useful in directing future research efforts, with particular regard to data collection.

DATA REQUIREMENTS FOR PERFORMANCE MODELING

While several model equations were developed during the course of this project for relating pavement distress to serviceability and performance, none of these can be considered as useful, verifiable models for performance prediction purposes. Several factors contributed to this result. For example, the project staff decided not to manipulate or discard the data made available to us for use in this project unless compelling evidence could be found to support such manipulation. Attempts were made to select specific subsets of data for use in the analysis, but these attempts were based on identifiable characteristics for the pavement sections involved, which were expected to have an impact on pavement performance. For example, a data base involving several hundred pavement sections could be divided into groups based on structural number, traffic levels, pavement type, layer thicknesses, or other identifiable pavement characteristics. Each subgroup was then analyzed individually. However, "out-liers" were not discarded without compelling reason, and groupings of pavement were not made arbitrarily in order to achieve better results. For instance, it was observed during the study that roughly 20 percent of the pavement sections in a particular data base exhibited similar serviceability and distress trends, and hence, could potentially be used to derive regression equations which would exhibit a good fit to the data, i.e., large values of R^2. However, these pavements involved a wide range of traffic, layer thicknesses, age, and so forth. No specific characteristics could be identified which would set this group apart from the other pavements in the data base. Indeed, many pavements of similar structure, traffic, etc., exhibited a variety of patterns of serviceability and distress development.

Several other factors influenced the modeling effort, such as limited project funding, delays experienced in obtaining data, and so forth. However, the single most important factor was found to be the inadequacy of the available data records.

During the conduct of this research, members of the project staff made contact with representatives of a dozen state highway agencies regarding pavement performance data. Data records were obtained from six of these agencies. Data records from the AASHO Road Test were also obtained and used in the study. In addition, an HRIS of the available

literature was conducted, and numerous research reports were reviewed. Four of these reports yielded data which were analyzed as a part of the research effort, while many others provided insight and direction to the research effort. Data were required from several sections of the Brampton Test Road.

Data from each of these sources were found to be inadequate for the purposes of this study. The major factors contributing to this inadequacy are discussed below. Of course, not all data sources exhibited all of the inadequacies listed below. In some cases, only a single factor was missing, while in others, several factors contributed to the inadequacy. However, in no case did a single data source prove entirely adequate.

The following major inadequacies were identified:

(1) Inadequate Time Record: One of the major problems in determining time dependent distress/serviceability relationships was the time scale involved in the data records. Many of the data records involved only one to three years of pavement distress and serviceability data. If the pavements involved had an average life of twenty years, such a limited sample would hardly provide an adequate basis for modeling.

(2) Failure to Record both Distress and Serviceability Data: Several states that were contacted regarding data for this project reported that distress and serviceability data were not available on the same data file, or simply were not available for the same pavement sections at all. Some states indicated that they could provide good records on particular distress types, but no corresponding record of serviceability; others reported that good serviceability records were available, but that conditions surveyed for distress records were no longer performed.

This inadequacy also relates to paragraph 1 above. In some cases, otherwise adequate data bases were found to have limited records for either serviceability or a particular distress type. For example, a data base which involved thirteen years of annual observations was found to have only nine to ten years of data for any particular distress type. At the same time, this data base recorded only a maximum of five annual serviceability measurements. In addition, two of the serviceability measurements were made during years for which no distress measurements were performed. Hence, the lack of distress and serviceability data, transformed an otherwise quite adequate data base in terms of time period and variables recorded, into a data base covering an inadequate time period.

(3) Lack of Maintenance Data: Very limited maintenance records were included in the data obtained for this study. One of the

data bases included specific maintenance records including seal coats and other major maintenance activities. However, the record provided was found to be only a partial record of the maintenance activities performed on the pavement sections involved. Another of the data bases recorded the area of patching in square feet. Presumably, however, other maintenance activities were carried out on the pavement sections. Yet another data base recorded only the sum of cracking plus patching. Thus, it was not possible to determine what portion of this total value reported was maintenance related.

(4) Omission of Key Variables: Those key distress variables believed to be most significant for this study are listed in Table 2. Of the variables listed for flexible and composite pavements, only rutting was recorded universally in the data examined. None of the selected rigid pavement variables were reported universally. In many cases, significant distress variables were lumped together or combined into a single index the value of which was reported in the data record.

    Regarding the serviceability or performance of pavements, most data sources reported a PSI on a 0 to 5 scale. However, some of the data sources reported only roughness or "bump count".

(5) Lack of Standardization of the Units: The only variable universally reported in the same units was rut depth, recorded in inches. Even so, the method for determining average rut depth varied among the data sources used. Other distress variables were recorded in a variety of units. For example, various forms of cracking were reported in units of square feet per thousand square feet of pavement surface, linear feet per thousand square feet, square feet of area affected, total length of cracking, number of cracks per section, and by distress level in terms of severity and extent.

Serviceability was generally recorded as PSI on a scale of 0 to 5. However, the equation used to calculate PSI varied from data base to data base. That is, no standard definition of PSI was adopted universally.

Given the wide variety of data sources examined for use in this project, it is likely that the problems encountered here are common to the vast majority of existing data sources. That is, data inadequacy is a very nearly universal problem. Therefore, it was felt that some guidelines should be provided for future data collection efforts. It should be emphasized that these guidelines apply specifically to data collected for modeling purposes; they may not be applicable to data collected for routine inventory purposes.

One approach to this would involve listing a straightforward set of solutions to the problems identified above. Thus, the lack of inadequate

time base could be solved by simply collecting data for longer periods of time, while the use of nonstandard units could be eliminated by adopting a federal standard for the measurement of various types of distress. However, beyond just correcting the obvious deficiencies, the only way to assure that meaningful modeling will be achieved is to design an experiment or experiments to incorporate all the relevant factors.

Consequently, project staff developed an "ideal" experiment designed to provide the data necessary for effective performance modeling. This ideal design is presented here as an aid to future research. It is not anticipated that this particular experiment will be performed, but it is felt that the considerations discussed here will provide guidance for future data collection efforts. The discussion deals only with flexible pavements, for example purposes. However, the same basic considerations carry over to rigid pavements, and a similar design could be constructed for the rigid case.

"Ideal" Experiment Design

The first step in the design of an experiment to collect data for pavement performance modeling is to identify the dependent variables (Y's) to be measured during the experiment. The list of important variables should likely include: (1) distress, (2) roughness, (3) deflection, and (4) skid resistance. Each of these basic variables may involve several sub-variables. For example, distress will undoubtedly involve rutting and fatigue cracking, as well as possibly low temperature cracking, bleeding, and other variables. It is desirable to limit this set of variables as much as possible without excluding important parameters.

The next step is to acknowledge the role of time. It must be pointed out that time should be regarded as a split-plot factor and not as a dependent variable or as a covariate which allows another term (a regression piece) to be placed in the analysis of variance model for the first stage in the analysis of the structure effects. The reason time should be regarded as a split plot factor in the ideal experiment is that it forces the investigator to obtain measurements throughout the entire experiment at fixed intervals of time for all treatment combinations. This, of course, eliminates the inadvertent confounding of time with particular treatment combinations and allows investigation of the interactions of time with all factors in the experiment. This probably is the single most important concept in the experiment design. Any departure from taking observations in a regularly scheduled time sequence will have confounding effects that cannot be completely accounted for in the analyses to follow. Of course, the shortcoming of any time variables is that the errors may be correlated and this will hurt good prediction models.

Next, a list of factors and levels to consider in the ideal experiment

must be developed. An example is given in Table 34. These factors are not supposed to be exhaustive or mandatory. However, it is felt that 15 factors will be sufficient to include all major influences on pavement performance.

It may be desirable to delete some factors, such as G or H, which overlap to some degree. Similarly, some factors may need further subdivision, such as M, which may require separate treatment of surface and sub-surface drainage.

If only 2 levels were run for each of the 15 factors, a design that would allow estimation of all main effects and two factor interactions (assuming three factor interactions are zero) is a 1/128 replication of the 15 factors in 8 blocks of 32 each. Of course, 8 blocks may not be necessary, and this design simply represents an "ideal" estimate made by project staff.

Such a design is given on page 70 and 72 of Reference 62[1] as Plan 128.15.32, 1/128 replication of 15 factors in 8 blocks of 32 units each. The Identity, Block confounding and Blocks for this design are reproduced in Figure 56.

Note that an appropriate block structure must be chosen, which may require a relabeling of the factors. For example, if the interaction (ABD) within the environmental category were to be used in blocks, then moisture, temperature, and freeze-thaw would be renamed "A", "B", and "D", respectively.

The design of Figure 56 is good for two-level interactions. However, it is anticipated that at least three levels will be needed in most if not all factors in order to investigate curvature (deviation from straight-line behavior). If curvature is needed in all the factors, a composite design could be run which would require a total of 31 more treatment combinations. Since only three levels would probably be used for each factor, we could represent the center point as 0 level and allow low = -1 and high = +1. Using this set of definitions for the levels, the 31 treatment combinations given in Table 35 should be added to the 256 in the original design, making 287 combinations. Of course there would need to be repeats of these 287 treatment combinations. It would be "ideal" to have a complete replicate of the whole experiment, but one may conceive of 13 repeats if engineering information were available on the experimental error and the 13 were used only to check the error.

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[1]"Fractional Factorial Experiment Designs for Factors at Two Levels", The Statistical Engineering Laboratory, National Bureau of Standards, Applied Mathematics Series 48.

TABLE 34 POTENTIAL FACTORS TO BE INCLUDED IN THE DESIGN OF AN IDEAL EXPERIMENT TO COLLECT DATA FOR PAVEMENT PERFORMANCE MODELING

CATEGORY	FACTOR
Structural	A. Surface thickness
	B. Surface type
	C. Base thickness
	D. Base type
	E. Subgrade strength
Environment	F. Moisture
	G. Temperature
	H. Freeze/thaw cycles
Load	J. Traffic or vehicle passes
	K. Percent trucks or equivalent axle load
Miscellaneous	L. Construction variability
	M. Drainage
	N. Maintenance, preventive
	O. Maintenance, corrective
	P. Geometry

Plan 128.15.32. 1/128 replication of 15 factors in S blocks of 32 units each.

Factors: A,B,C,D,E,F,G,H,J,K,L,M,N,O,P.

I = ABEGN = ACEFNP = BCFGP = DEFGO = ABDFNO = ACDGNOP = BCDEOP = ADHKO
= BDEGHKNO = CDEFHKNOP = ABCDFGHKOP = AEFGHK = BFHKN = CGHKNP
= ABCEHKP = BCHJNOP = ACEGHJOP = ABEFHJO = FGHJNO = BCDEFGHJNP = ACDFHJP
= ABDGHJ = DEHJN = ABCDJKNP = CDEGJKP = BDEFJK = ABFGJKN = ABCEFGJKNOP
= CFJKOP = BGJKO = AEJKNO = ABKLOP = EGKLNOP = BCEFKLNO = ACFGKLO
= ABDEFGKLP = DFKLNP = BCDGKLN = ACDEKL = BDHLP = ADEGHLNP = ABCDEFHLN
= CDFGHL = BEFGHLOP = AFHLNOP = ABCGHLNO = CEHLO = ACHJKLN = BCEGHJKL
= EFHJKLP = ABFGHJKLNP = ACDEFGHJKLNO = BCDFHJKLO = DGHJKLOP
= ABDEHJKLNOP = CDJLNO = ABCDEGJLO = ADEFJLOP = BDFGJLNOP = CEFGJLN
= ABCFJL = AGJLP = BEJLNP = CDGHJMO = ABCDEFHJMNO = ADEFGHJMNOP
= BDFHJMOP = CEFHJM = ABCFGHJMN = AHJMNP = BEGHJMP = ACGJKM = BCEJKMN
= EFGJKMNP = ABFJKMP = ACDEFJKMO = BCDFGJKMNO = DJKMNOP = ABDEGJKMOP
= BDGMNP = ADEMP = ABCDEFGM = CDFMN = BEFMNOP = AFGMOP = ABCMO
= CEGMNO = ABGHKMNOP = EHKMOP = BCEFGHKMO = ACFHKMNO = ABDEFHKMNP
= DFGHKMP = BCDHKM = ACDEGHKMN = ABCDGHJKLMP = CDEHJKLMNP
= BDEFGHJKLMN = ADFHJKLM = ABCEFHJKLMOP = CFGHJKLMNOP = BHJKLMNO
= AEGHJKLMNO = BCGJLMOP = ACEJLMNOP = ABEFGJLMNO = FJLMO = BCDEFJLMP
= ACDFGJLMNP = ABDJLMN = DEGJLM = ADGKLMNO = BDEKLMO = CDEFGKLMOP
= ABCDFKLMNOP = AEFKLMN = BFGKLM = CKLMP = ABCEGKLMNP = GHLMN
= ABEHLM = ACEFGHLMP = BCFHLMNP = DEFHLMNO = ABDFGHLMO = ACDHLMOP
= BCDEGHLMNOP.

Block confounding: ABD, ACF, BCDF, ABCE, CDE, BEE, ADEF

Blocks only: All two-factor interactions are measurable.

Blocks

1

(1)	bcdfgjmo	acdghk	abfhjkmo	acdgjl	abflmo	hjkl	bcdfghklmo
cefhjklmn	bdcghklno	adefgjlmn	abcelno	adcfghkmn	abcehjkno	cefmn	bdcgjno
adefhjlop	abceghlmp	cefgjklop	bdeklmp	cefghop	bdchjmp	adcfkop	abcegjkmp
acdkmnop	abfgjknp	ghmnop	bcdfhjnp	gjklmnop	bcdjklnp	acdhjlmnop	abfghlnp
2	3	4	5	6	7	8	

Figure 56 "Ideal" Experiment Design Using
15 Factors in 8 Blocks (After Reference 62)

TABLE 35 ADDITIONAL TREATMENT COMBINATIONS NEEDED TO INVESTIGATE CURVATURE IN THE "IDEAL" EXPERIMENT

FACTOR

	A	B	C	D	E	F	G	H	J	K	L	M	N	O	P
1.	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2.	0	0	0	0	0	0	0	0	0	0	0	0	0	0	+1
3.	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1
4.	0	0	0	0	0	0	0	0	0	0	0	0	0	+1	0
5.	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	0
6.	0	0	0	0	0	0	0	0	0	0	0	0	+1	0	0
7.	0	0	0	0	0	0	0	0	0	0	0	0	-1	0	0
8.	0	0	0	0	0	0	0	0	0	0	0	+1	0	0	0
9.	0	0	0	0	0	0	0	0	0	0	0	-1	0	0	0
10.	0	0	0	0	0	0	0	0	0	0	+1	0	0	0	0
11.	0	0	0	0	0	0	0	0	0	0	-1	0	0	0	0
12.	0	0	0	0	0	0	0	0	0	+1	0	0	0	0	0
13.	0	0	0	0	0	0	0	0	0	-1	0	0	0	0	0
14.	0	0	0	0	0	0	0	0	+1	0	0	0	0	0	0
15.	0	0	0	0	0	0	0	0	-1	0	0	0	0	0	0
16.	0	0	0	0	0	0	0	+1	0	0	0	0	0	0	0
17.	0	0	0	0	0	0	0	-1	0	0	0	0	0	0	0
18.	0	0	0	0	0	0	+1	0	0	0	0	0	0	0	0
19.	0	0	0	0	0	0	-1	0	0	0	0	0	0	0	0
20.	0	0	0	0	0	+1	0	0	0	0	0	0	0	0	0
21.	0	0	0	0	0	-1	0	0	0	0	0	0	0	0	0
22.	0	0	0	0	+1	0	0	0	0	0	0	0	0	0	0
23.	0	0	0	0	-1	0	0	0	0	0	0	0	0	0	0
24.	0	0	0	+1	0	0	0	0	0	0	0	0	0	0	0
25.	0	0	0	-1	0	0	0	0	0	0	0	0	0	0	0
26.	0	0	+1	0	0	0	0	0	0	0	0	0	0	0	0
27.	0	0	-1	0	0	0	0	0	0	0	0	0	0	0	0
28.	0	+1	0	0	0	0	0	0	0	0	0	0	0	0	0
29.	0	-1	0	0	0	0	0	0	0	0	0	0	0	0	0
30.	+1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
31.	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Analyses

Let us assume that all measurements of roughness (y_1), deflection (y_2), skid resistance (y_3), rutting (y_4) and fatigue cracking (y_5) have been taken over the time intervals desired for all 300 treatment combinations. There may be as many time intervals as desired.

1. Analysis at each time period

   One could run an analysis of variance (ANOVA) on the 256 treatment combinations, plus the 13 repeats for pure error at each time period for each of the five y's (assuming appropriate transformations were made to make all variables normally distributed as follows:

Source	df
Blocks	7
δ	0
Main Effects	15
Two Factor Interactions	105
Residual	128
Pure error	13
Total	268

After finding out which two factor interactions were significant, one could use all 300 observations and run a multiple regression on each dependent variable, y_i ($i = 1,2,\ldots,5$) as follows:

$$y_i = \beta_0 + \beta_1 x_1 + \ldots + \beta_{15} x_{15} + \beta_{1,1} x_1^2 + \ldots + \beta_{15,15} x_{15}^2 + \ldots$$

all two factor interactions significant in the ANOVA for the ith dependent variable + residual + pure error.

After the engineers were satisfied with the models (here we say 5 models, but it could be more or less), one may look for an index to put all these variables together as an "overall performance" index.

2. Analysis over Time (where time is a split-plot factor)

   Run an ANOVA on each y, say over 11 time periods, to get a number to show in the ANOVA (the number of time periods could be greater or smaller). The ANOVA would be the following:

Source	df
Blocks	7
δ_1	0
(A thru P) Main Effects (M.E.)	15
(A thru P) Two Factor Interactions (2f.i.)	105
Residual	128
Pure error	13
δ_2	0
Time (T)	10
T x Blocks	70
T x M.E.	150
T x 2f.i.	1050
T x Residual	1280
T x Pure error	130

The most important part of this ANOVA is to find out if the interpretation of T x Pure error mean square is the same order of magnitude as Pure error mean square. This concept is covered at the end of Chapter 7 of Reference 63[1].

The next important part, given that the first one shows these errors are the same size, is T x Residual vs. Residual. Next after that T x 2f.i. vs. Two Factor Interactions and finally T x M.E. vs. Main Effects.

If the errors (Pure and Residual) can all be pooled then an overall regression analysis may be run for each y_i as follows:

$$y_i = \beta_1 x_1 + \ldots + \beta_{15} x_{15} + \beta_{1,1} x_1^2 + \ldots + \beta_{15,15} x_{15}^2$$

+ significant two factor interactions of the 15 factors,
+ significant time and interactions of time effects,
+ residual + pure error.

Again after the engineers are satisfied with the models, they may seek an index for "overall performance" using the y-variables.

If it turns out that the errors, Pure and T x Pure, cannot be pooled, there may be correlation of the errors. To examine the effects of this condition in the factorial part, one may use the procedure given on page 166 of Anderson and McLean (Ref. 63). In this case, one calculates the sums of squares as given above, but the degrees of freedom for the sources are used as follows:

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[1]V.L. Anderson and R.A. McLean, "Design of Experiments: A Realistic Approach", Marcel Dekker, Inc., N.Y., 1974.

Source	df
Time (T)	1
T x Blocks	7
T x M.E.	15
T x 2f.i.	105
T x Residual	128
T x Error	13

If the results of all the F-tests are the same as for the previous tests using 10 times the degrees of freedom (df), one need not be concerned about correlated errors. If, however, there are major differences, care must be taken in the interpretation and use of the variables in the regression equations. There is no clear-cut way to ideally obtain all the information due to time if the errors are too highly correlated.

Other Design Approaches

There are many types of designed experiments that could be used for this problem, but the most efficient one seems to be the one discussed thoroughly above.

If one wants to investigate three factor interactions, an entirely different design of experiment must be made. It would require many more treatment combinations than the design presented here.

If curvature must be examined for all combinations of the 15 factors, a fractional factorial of 3^{15} may be needed. The number of treatment combinations required for this type of design is quite large, even if three factor interactions are assumed zero.

PERFORMANCE MODEL REQUIREMENTS FOR PAVEMENT MANAGEMENT

Performance models are used in two distinct contexts as a part of pavement management, depending upon the pavement management level involved. At the project level, fairly detailed and specific models are required for predicting the performance expected for an individual pavement section. At the network level, general or "average" prediction models are required to provide estimates of the expected performance for a typical pavement or class of pavement. Accordingly, quite distinct modeling approaches are indicated for these two different modeling needs (Ref. 85)[1].

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[1]Hudson, W.R., R. Haas, and R.D. Pedigo, "Pavement Management System Development", NCHRP Report No. 215, Transportation Research Board, November 1979.

At the project level, considerable information will be available regarding the pavement structure, the current and expected traffic, current and past distress measurements, deflections, and so forth. The prediction model used must be able to predict specific values for the performance of the given section in an accurate and reliable fashion. Thus, a fairly accurate prediction model specific to the individual conditions appropriate to a single project is needed. The research effort in this project indicates that time dependent distress/serviceability relationships of this type are extremely difficult to derive. The primary reason for the difficulty lies in the lack of adequate data records covering a sufficiently long time period.

However, another potentially viable approach is available for project level modeling. Such an approach would involve predicting future distress, and then relating distress to serviceability in a time independent model. Distress prediction models for various distress types are available, and have been discussed and evaluated in this report. These are mechanistically based models, and they require a large number of inputs for their operation. Such models will therefore be useless at the network level, where detailed information is not available. However, at the project level, it is feasible to obtain sufficient data to provide input to one or more of these mechanistic models. The output would be a prediction of one or more future distress levels. It then only remains to relate these future distresses to serviceability. The results of this research indicate that, given the current state of available data, it is in fact more feasible to relate distress to serviceability directly, with no consideration of time as a variable.

In this approach, the distress/serviceability relationship required at the project level is time independent. Several such models were developed during the course of this investigation, although none of these can be considered widely applicable in their present form. It is clear, however, that the models developed for small groups of similar pavements are more reliable than those developed for a large data base. Therefore it seems likely that by carefully selecting several classes of similar pavements, one could produce time independent distress/serviceability relationships for each class which would be reliable enough for project level pavement management use. Such models would, of necessity, be very limited in applicability; that is, each model would apply to only a very small class of pavements, so that each agency would require several such models in order to predict performance for a variety of pavement projects.

For network level applications, such an approach would be less viable. The mechanistic distress models require information of a much too detailed character for network level applications. Even if such data were available, the amount of time required for such analysis at the network level would be prohibitive. On the other hand, the results of this research indicate that the formulation of direct time dependent

distress/serviceability relationships is probably not feasible either in the absence of a long-term data record. Thus, the development of direct distress/serviceability relationships for network level pavement management is not likely to be feasible for a number of years.

There is, however, an alternative approach. The result of this project indicates that Markovian or Bayesian techniques may be used to develop performance prediction models which utilize distress/serviceability relationships only indirectly. Since only an average performance prediction for any pavement section is required at the network level, the lack of adequate data is not as troublesome as it is for project level modeling. Bayesian or Markovian techniques are particularly applicable for this case, and in fact these techniques may be implemented in situations where little or no objective data are available.

Such a network level performance prediction model based on purely indirect distress/serviceability relationships is presented in the following section.

STOCHASTIC SERVICEABILITY DETERIORATION MODEL FOR RIGID PAVEMENTS

During the course of this investigation, it became clear that the lack of adequate data would have a great impact on the development of distress/ performance relationships for use in performance prediction for pavement management. Consequently, project staff began to consider alternative performance prediction methods based on indirect relationships between distress and performance. Modeling efforts based on Markov Processes and Bayesian Analysis were undertaken. These techniques are widely discussed in the literature (Refs. 64[1], 65[2], 66[3], 67[4]), and have been applied in several areas of pavement modeling (Refs. 68[5],

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[1]Hillier, F.S. and G.J. Lieberman, Introduction to Operations Research, Holden-Day, Inc., San Francisco, 1967.

[2]Martin, J.J., Bayesian Decision Problems and Markov Chains, Wiley, New York, 1967.

[3]Martiz, J.S., Empirical Bayes Methods, Methuen and Co., Ltd., London, 1970.

[4]Stark, R.M. and R.L. Nicholls, Mathematical Foundations for Design: Civil Engineering Systems, McGraw Hill, New York, 1972.

69[1], 70[2], 71[3]).

As a part of the modeling effort discussed in Chapter 5, serviceability histories for rigid pavement sections from loops 3, 4, 5, and 6 of the AASHO Road Test were examined. It was observed that roughly seven of every ten sections that reached terminal serviceability during the Road Test exhibited a characteristic serviceability pattern: a long period of nearly constant serviceability followed by a precipitous drop near the end of the service life. This pattern is evident in Figure 57, which illustrates serviceability plotted against service life for twenty of the rigid pavement sections which "failed" during the Road Test. "Service Life" is defined here as the length of time between the beginning of the test and the time at which a particular section reached terminal serviceability. Thus, the service life scale used in the figure is a time scale, normalized by the total length of time that each pavement section was in service. Another way of saying this is that time is reported on the figure as a fraction of the total service life for each pavement section.

The pattern illustrated in Figure 57 was found for pavement sections with slab thickness ranging from 2.5 inches (6.4 cm.) to 11 inches (28 cm.), applied axle load ranging from 6 kip (27 kN) single axles to 48 kip (213 kN) tandem axles, pumping scores ranging from 500 to 60,000, and a similar range of other parameters. Thus, it was felt that this pattern could serve as the basis for a fairly general, widely-applicable performance model.

There are a number of functional forms which could be used to reproduce the general shape illustrated in Figure 57. In the hope of obtaining a model which could be adapted to a variety of pavement types and structures, the following general form was chosen for use in this

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[5]Smith, W.S., F.N. Finn, C. Saraf, and R. Kulkarni, "A Bayesian Approach Applied to Prediction of Pavement Distress", paper presented at the Annual Meeting of the Transportation Research Board, January 1978.

[1]Finn, F.N., W.S. Smith, C. Saraf, and R. Kulkarni, "Bayesian Analysis Methodology for Verifying Recommendations to Minimize Asphalt Pavement Distress", Draft Final Report, NCHRP Project 9-4A, January 1978.

[2]Karan, M.A., Municipal Pavement Management System, Thesis, University of Waterloo, 1977.

[3]Smith, W.S., A Flexible Pavement Maintenance Management System, Dissertation, University of California, Berkeley, 1974.

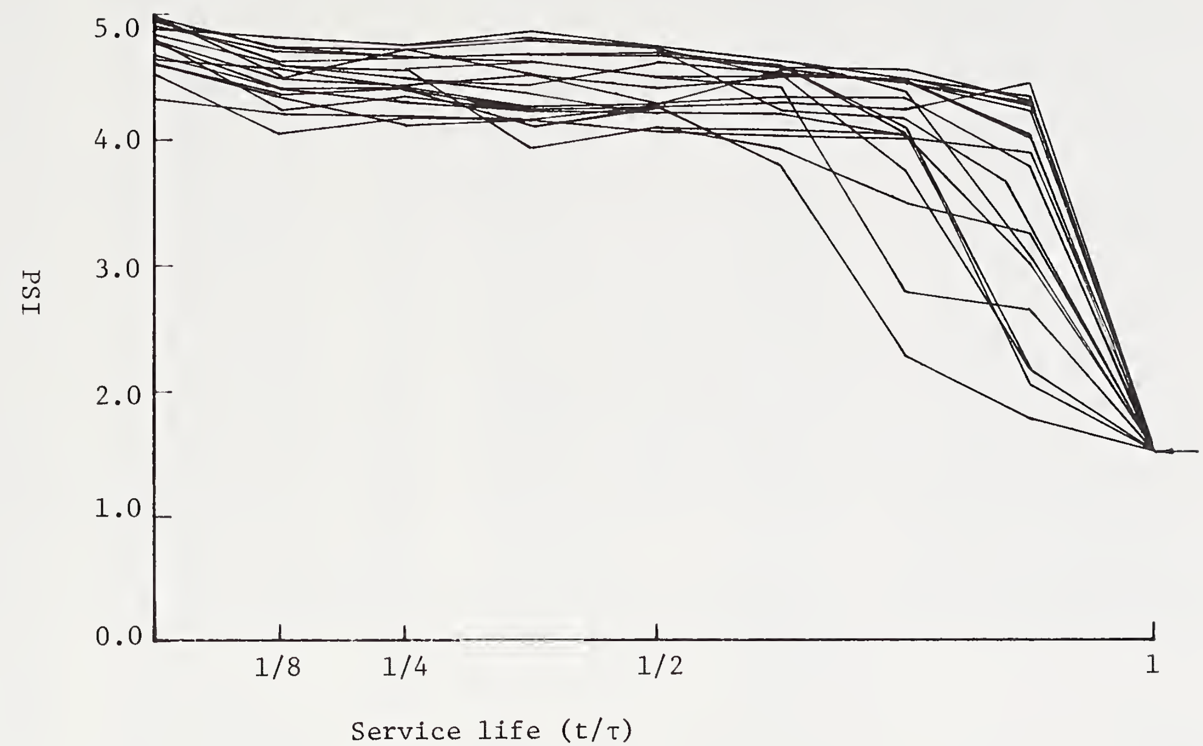

Figure 57 Serviceability History Using a Normalized Service Life Time Scale, AASHO Road Test Rigid Pavements

project:

$$\overline{PSI}\,(t) = C_1 + \frac{C_2}{[e^{\beta(t/\tau - 1)} + 1]} \qquad (18)$$

where:

$\overline{PSI}\,(t)$ = average predicted serviceability at time t,
C_1, C_2 = constants, determined from initial and terminal serviceabilities,
β = parameter, presumably dependent upon pavement structure, load, and environment, that determines the shape of the predicted serviceability history curve,
and τ = the expected service life of the pavement (time, in years, from beginning of traffic to terminal serviceability).

By adjusting the values of coefficients τ and β Equation 18 can be made to reproduce the shape of any of the serviceability patterns for flexible or rigid pavements identified in Chapter 4. The values chosen here to reproduce the behavior of Figure 56 are shown in the following equation:

$$\overline{PSI}\,(t) = -1.5 + \frac{6.0}{[e^{10(t/\tau - 1)} + 1]} \qquad (19)$$

This equation is plotted as the solid curve in Figure 58. Notice that the parameter τ need not be specified in order to compute serviceability at any fraction of the expected service life. However, the value of τ must be fixed in order to translate this fraction of service life into actual time in years.

Equation 19 may be applied directly to the prediction of pavement performance in the following manner:

1. Chose an expected service life, τ, for the pavement or the class of pavements for which the serviceability is to be predicted. This will provide a means for translating between pavement age or time in years and the appropriate fraction of service life used in equation 19. For example, the design life of a pavement section maybe taken as an estimate of its expected service life.

2. For any value of time, t, for which the serviceability is to be predicted, calculate the fraction of service life. This is done by dividing the time t, expressed in years since the section was opened to traffic, by the service life, τ, expressed in years.

3. Insert the calculated value of fraction of service life (t/τ)

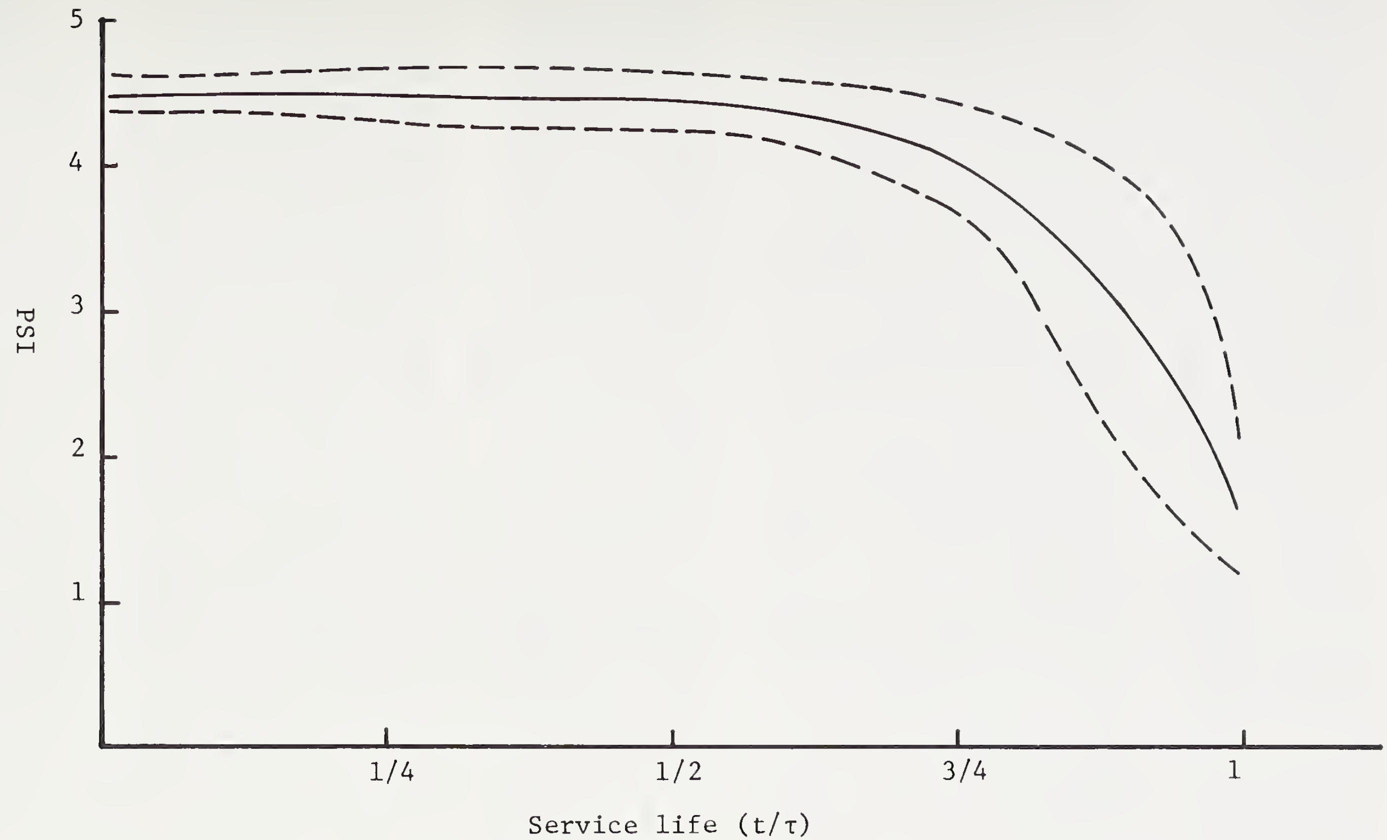

Figure 58 Predicted Serviceability History Based on Stochastic Version of Equation 19

into Equation 19 and compute the desired serviceability. Alternatively, enter the graph of Figure 58 at the calculated fraction of service life, and proceed vertically until the solid line is encountered. Then proceed horizontally to intersect the PSI axis. The point of intersection is predicted serviceability for the pavement section.

Of course, some variability is observed in Figure 57 for the behavior of individual pavement sections. In order to account for this, a stochastic feature was added to the prediction given by Equation 19. In this approach, the PSI predicted by Equation 19 is to be interpreted as a mean serviceability index for the pavements in question. Variations about this mean are incorporated by defining an artificial variance or standard deviation. Estimated values for such a standard deviation were derived from the magnitude of the variation observed in Figure 57. These values were used to construct the dashed lines in Figure 58, which represent the mean value $\pm$ twice the artificial standard deviation. Using these dashed curves, step 3 above may be repeated to obtain estimates of the maximum and minimum PSI to be expected.

There are a number of problems with the straightforward application of Equation 19 in this manner. For example, major maintenance activities may be carried out, severe environmental conditions may be encountered, traffic levels may change dramatically, and so forth. No procedure is given for incorporating these effects, and, in addition, no information is provided regarding exactly how likely it is that the predicted mean serviceability will be achieved or that the actual serviceability will lie between the two dashed curves in the figure.

In order to address some of these problems, the predicted serviceability history of Equation 19 and Figure 58 was incorporated in a Markovian framework. In such an approach, the pavement is described as being in certain "state" at any given time, and the probability that pavement will undergo a transition to each other possible state within a fixed short period of time is specified. Such a model is conveniently expressed in Matrix notation, with transition probabilities arrayed in a square matrix, and possible pavement states listed in a single column matrix. In this example, pavement states are specified in terms of pavement serviceability, but other significant variables may be incorporated in the descriptions of pavement states (Refs. 70, 71).

Twenty possible pavement states, or serviceability values, were selected for use in this example. These are listed in Table 36. Nominal serviceability values are specified for each state. Once these states have been specified, transition probabilities between states may be calculated using Equation 19 and Figure 58. A set of transition probabilities which effectively reproduce the behavior illustrated in Figure 58 is listed in Table 37. In developing this transition matrix, the time interval between transitions was fixed at 1/100 of the expected

Table 36 LIST OF POSSIBLE PAVEMENT SERVICEABILITY STATES FOR MARKOV MODEL

STATE	NOMINAL PSI
1	5.0
2	4.9
3	4.8
4	4.7
5	4.6
6	4.5
7	4.4
8	4.3
9	4.2
10	4.0
11	3.8
12	3.6
13	3.4
14	3.2
15	3.0
16	2.7
17	2.4
18	2.1
19	1.8
20	1.5

TABLE 37 TRANSITION PROBABILITY MATRIX FOR MARKOV EXAMPLE PROBLEM

TRANSITIONS TO THESE PSI STATES

TRANSITIONS FROM THESE PSI STATES	5.0	4.9	4.8	4.7	4.6	4.5	4.4	4.3	4.2	4.0	3.8	3.6	3.4	3.2	3.0	2.7	2.4	2.1	1.8	1.5
5.0			1.0																	
4.9			1.0																	
4.8			.010	.030	.100	.220	.280	.220	.120	.020										
4.7			.002	.026	.153	.448	.210	.095	.053	.013	.001									
4.6			.002	.024	.145	.421	.202	.099	.074	.030	.004									
4.5			.002	.023	.137	.399	.193	.097	.081	.049	.017	.003	.001							
4.4			.002	.023	.137	.399	.193	.097	.081	.049	.017	.003	.001							
4.3			.002	.021	.127	.371	.179	.091	.079	.058	.036	.021	.011	.004	.001					
4.2			.002	.021	.119	.272	.184	.108	.098	.074	.048	.034	.022	.012	.006	.001				
4.0			.003	.012	.042	.097	.139	.138	.150	.120	.082	.063	.052	.038	.032	.019	.008	.003	.001	
3.8				.002	.011	.030	.061	.088	.153	.159	.116	.090	.078	.061	.059	.045	.026	.013	.006	.003
3.6						.002	.006	.017	.055	.134	.148	.128	.112	.091	.094	.083	.058	.036	.021	.016
3.4						.002	.005	.014	.045	.108	.119	.103	.092	.077	.084	.084	.073	.060	.053	.085
3.2								.003	.015	.056	.096	.111	.107	.091	.100	.100	.087	.072	.060	.102
3.0								.001	.006	.024	.056	.089	.109	.106	.117	.118	.102	.085	.070	.119
2.7									.001	.011	.029	.058	.091	.107	.130	.137	.119	.098	.082	.139
2.4										.005	.015	.032	.066	.089	.130	.153	.139	.115	.096	.162
2.1										.002	.008	.017	.044	.069	.112	.157	.156	.134	.112	.190
1.8										.002	.008	.017	.044	.069	.112	.157	.156	.134	.112	.190
1.5											.002	.007	.024	.045	.090	.147	.164	.154	.133	.233

service life of the pavement. Thus, for a service life of 20 years, 5 transitions per year are incorporated in Table 37. At each transition, the pavement may remain in its current state or enter another state (improve or deteriorate).

Predictions of pavement serviceability are carried out in the following manner: First, the initial state of the pavement is specified. This is done in terms of the probability that the pavement has a specific serviceability level at the initial time. If the pavement is known to have PSI = 4.5 exactly, then the probability that the pavement exists in state number 6 of Table 36 is 1.0, and the probabilities for all the other states are 0. However, in the general case, the serviceability of the pavement can be specified only within some limit; say for example, a mean serviceability of 4.5, with standard deviation 0.1. The initial state specification for this case is given in Table 38. Such a state is called a "mixed state". The probability values in this case may be thought of as expressing the likelihood that a repeat measurement of PSI would yield the nominal PSI value associated with each state in the table.

The state of the pavement at future times is calculated by multiplying the initial state by the transition matrix. The state of the pavement after one transition is obtained by multiplying the initial state by the transition matrix once. For two transitions, the multiplication is carried out twice, and so forth. In this example, the state of the pavement at the mid-point of its service life would require 50 such multiplications. Thus, in actual practice, it might be advisable to use fewer transition per year, perform the calculations on a computer, or both.

In this approach, the procedure for obtaining predictions for future PSI values is fixed: multiply the existing pavement state by the transition matrix. However, the formalism allows modification of pavement states and transition probabilities to account for such effects as resurfacing, accelerated pavement distress, increased traffic, and so forth. If the observed state of the pavement is found to differ from the predicted state, then the observed state may be substituted into the matrix multiplication process. Such a difference in observation and prediction could occur, for example, if the pavement were resurfaced after, say, two-thirds of the expected service life.

Of course, resurfacing, the occurrence of accelerated distress, or dramatic increase in traffic volume could affect the expected rate of pavement deterioration as well as the current state of the pavement. These effects may also be incorporated in this formalism by adjusting the transition probabilities, or replacing Table 37 with a transition matrix calculated on the basis of a faster or slower rate of deterioration. Thus, one transition matrix could be specified for the original pavement, another for overlayed pavements, etc. This is the approach taken in Reference 71.

TABLE 38 INITIAL STATE SPECIFICATION FOR $\overline{PSI} = 4.5$, $\sigma = 0.1$

STATE	NOMINAL PSI	PROBABILITY
1	5.0	
2	4.9	0
3	4.8	0.006
4	4.7	0.061
5	4.6	0.241
6	4.5	0.383
7	4.4	0.241
8	4.3	0.061
9	4.2	0.007
10	4.0	0
11	3.8	
12	3.6	
13	3.4	
14	3.2	
15	3.0	
16	2.7	
17	2.4	
18	2.1	
19	1.8	
20	1.5	

Of course, if several different transition matrices are required for each pavement section to be studied, quite a large number of calculations would be required. However, the behavior illustrated in Figure 57 indicates that a wide variety of pavements may be represented by a single matrix. Thus, an agency could have one transition matrix, say, for each functional class of new pavement. An additional matrix could be specified for each functional class for overlayed pavements. If the agency must deal with pavements in widely differing environments, then a different set of matrices could be required for each different region. Thus, there is a reasonable expectation that twenty or thirty transition matrices could be sufficient to provide reasonable serviceability predictions for most or all of the pavements for which an agency is concerned. Such predictions would, of necessity, represent the "average" expected serviceability pattern for the pavement functional class, environment, etc., rather than the best estimates for an individual pavement section. Hence, such an approach is expected to be most useful for network level pavement management applications.

TOWARD AN OVERALL MEASURE OF PAVEMENT PERFORMANCE

The term "performance", as applied to pavements, has come to mean an accumulated measure of pavement serviceability over a period of time or accumulated load. However, the original definition of pavement performance, quoted earlier in this report, is: Performance is a measure of the accumulated service provided by a facility; i.e., the degree to which a pavement fulfills its purpose (Ref. 1).

Considerable disagreement was found among project staff, advisory panel members, state highway department engineers, and our colleagues in the pavement field, as to whether the PSI record for a pavement section represented a good measure of its performance. Without attempting to resolve whether PSI is a good measure of serviceability, or whether the roadway user should be the final judge of the adequacy of the pavement, consideration is given here to the need for an overall measure of adequacy or performance of pavements.

The need for such an overall performance measure is most evident to agencies that have begun the development of pavement management systems. Systematic pavement management requires that comparison be made among alternative plans of action in order to determine the most appropriate strategy for maintaining and rehabilitating a pavement section or a pavement network. Such comparisons are most readily carried out in terms of a single variable or "objective function" which represents the overall performance expected under each plan of action. If it necessary to compare several variables in order to determine the adequacy of each plan of action, the procedure for selecting the most desirable strategy becomes extremely complex. Some engineers and highway departments are willing to accept serviceability as the objective function for pavement management. Others feel that cracking, deflections, and other pavement

attributes must be considered when evaluating alternative actions. If several factors are to be involved in the comparison of alternative strategies, then a method of combining these factors into a single overall performance indicator is required in order to carry out such comparisons efficiently and economically.

Utility Theory

One method for achieving such an overall performance measure involves the application of utility theory (Refs. 5, 64, 67). In such an approach, numerical values from 0 to 1, called utilities, are assigned to each of the pavement attributes to be combined. This has the affect of putting all of the pavement attributes on the same scale, so that they may be simply combined. Thus, to avoid adding apples and oranges, one converts both apples and oranges to a common scale: fruit. The combined utility for all attributes of a given pavement section is calculated as follows:

$$u(\underline{x}) = \sum_{i=1}^{n} k_i u_i(x_i) \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \quad (20)$$

where:

$u(\underline{x})$ = multiattribute utility, scaled from 0 to 1.
$u_i(x_i)$ = individual utility for the ith attribute, x_i, scaled from 0 to 1.
k_i = scaling constant representing the relative weight of the ith attribute, such that $\Sigma\, k_i = 1$.

As an example of how this combined utility may be obtained, suppose that two attributes, serviceability and cracking, are to be combined. First it is necessary to establish individual utility functions $u_i(x_i)$ for serviceability (i=1) and cracking (i=2). This may be done by soliciting expert opinion through specific interview procedures (Ref. 5). The results of such an interview, combined with required analytical procedures, would produce relationships like those shown in Figure 59.

In Figure 59 the assessors have indicated that significantly greater benefits are achieved by going from PSI to 1.0 (u_1=0.25) to 2.0 (u_1=0.55) than by going from 3.5 (u_1=.85) to 4.5 (u_1=.97). Thus, the decision maker would be more willing to spend funds to improve a poor pavement than a good pavement. This, of course, is obvious; however, the technique provides a means by which such questions can be answered numerically. Similar information can be obtained from Figure 59 applicable to cracking.

Values for k_1 and k_2 must next be chosen to represent the relative impact of the two attributes. This may also be done through personal assessment. Suppose, for example, that the values k_1 = 0.7 and k_2 = 0.3 are established in this manner. Then, using Figure 59, the combined

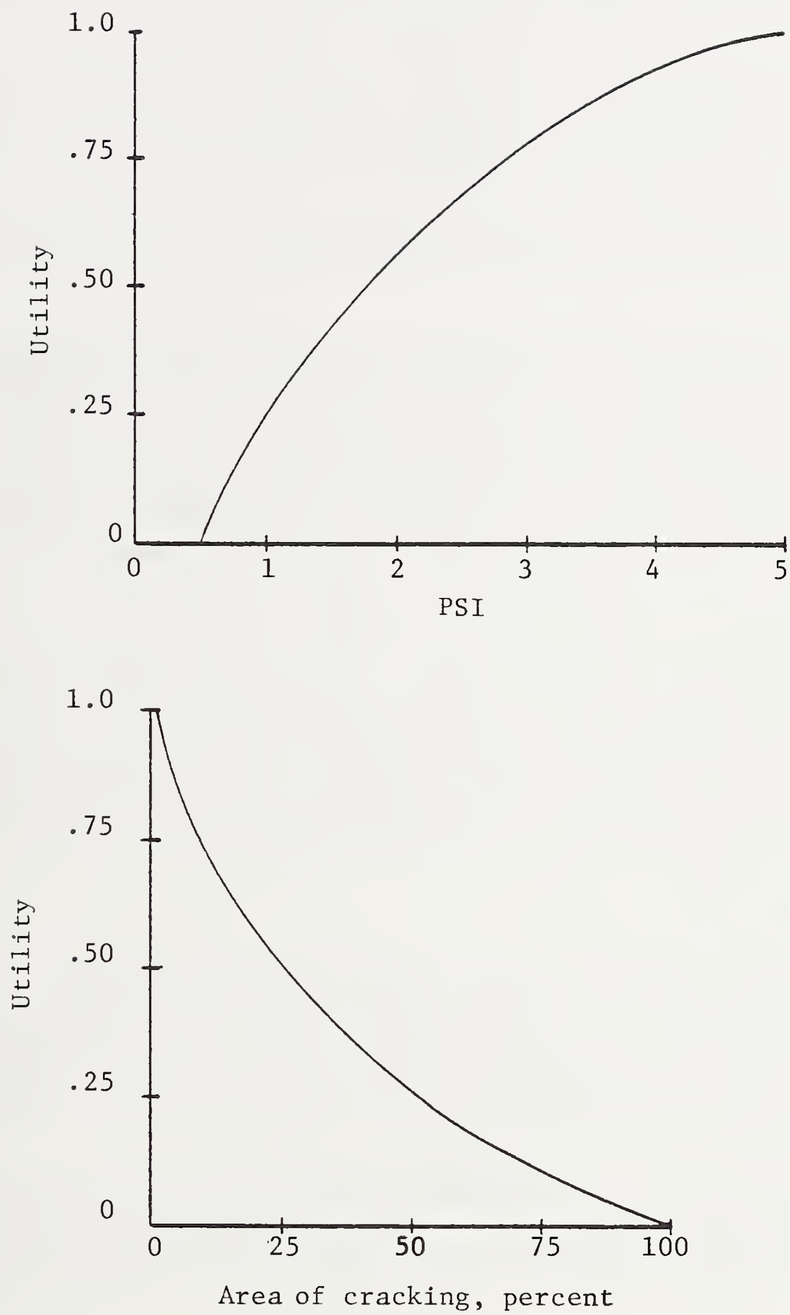

Figure 59 Relationship between utility value and cracking or serviceability for example problem

multi-attribute utility of a pavement section with present serviceability index of 3.0 and area of cracking of 50 percent would be obtained as follows:

$$\text{PSI} = 3.0;\; u_1 = 0.78$$

$$\text{Area of Cracking} = 50\%;\; u_2 = 0.25$$

$$\begin{aligned} u\,(\underline{x}) &= 0.7 * 0.78 + 0.3 * 0.25 \\ &= 0.456 + 0.075 \\ &= 0.621 \end{aligned}$$

If the area of cracking were to be reduced to 0 percent the combined utility would be 0.846; 0.546 for serviceability plus 0.3 for cracking.

The utility theory approach offers several advantages. First, any number of pavement attributes may be combined in such a fashion. The resulting multi-attribute utility always has a value between 0 and 1, making further rescaling unnecessary. The numerical value of the multi-attribute utility may be loosely interpreted as the degree to which a pavement section approaches perfection, expressed as a decimal fraction. Finally, the multi-attribute utility is expressed on a scale which allows differences in utilities to be directly compared from section to section.

The principle disadvantage of the utility theory technique is that the final multi-attribute utility contains no information regarding the individual attributes from which it was constructed. That is, many different combinations of serviceability level and extent of cracking could lead to the same overall utility. Thus, given only the overall utility, it would be impossible to calculate the serviceability or extent of cracking for the pavement section.

Unique Sums

An alternative way of combining pavement attributes is offered by the method of unique sums (Ref. 72)[1]. In this approach, pavement attributes are assigned numerical values in such a way that, when these values are added together, the sum is unique; that is, given only the total, it is possible to recover the specific values for each of the individual attributes involved.

Suppose that it is desired to combine the pavement attributes A (serviceability), B (cracking), and C (rut depth). A fixed number of levels is first established for each of these attributes. In this

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[1]Anderson, Olle, "Features of the Swedish Triennial Road Inventory System", Bulletin 1979:10, Department of Highway Engineering, Royal Institute of Technology, Stockholm, Sweden, 1979.

example, four levels are available for each attribute. For convenience, these are labeled as levels 1 (excellent), 2 (acceptable), 3 (marginal), and 4 (unacceptable). Thus, for attribute A, serviceability, values from PSI = 3.6 to 5.0 could be assigned to level 1, values from 2.8 to 3.5 to level 2, 2.0 to 2.7 to level 3, and 1.9 or below to level 4. Similar categorizations could be made for each of the attributes.

Once this has been done, the next step is to assign numerical values to each level for each attribute. These values must be chosen in such a way that any value from one of the four levels of attribute A, plus the value from any of the four levels from attribute B, plus the value associated with any of the four levels of attribute C yields a unique answer. A set of numerical values which produce such unique sums can be found in Table 39.

As an example of the application of this technique, consider a pavement section which exhibits level 2 of attribute A, level 1 of attribute B, and level 3 of attribute C. Adding together the corresponding values shown in Table 39, the unique sum for this pavement is 18 + 17 + 15 = 50. It is not possible to generate the number 50 from any other combination of levels for attribute A, B, and C. Thus, given that the unique sum for a particular pavement is 50, its serviceability must fall in level 2, its cracking must lie in level 1, and it must have level 3 rutting. The information about individual pavement attributes is consequently preserved in the unique sum, and can be reconstructed quite easily at any later date.

This technique can in principle be extended to include any number of pavement attributes and levels for each attribute. In practice, the numerical values required for a large number of levels and attributes can become quite large. This can be seen in Table 40, which gives a unique sum categorization for four attributes at four levels. Of course, decimals or fractions may be used to limit the numerical range.

The major advantage of this approach is the preservation of information regarding the individual pavement attributes. The primary disadvantage is that the numerical value of the unique sum is reported on a highly non-linear scale. Thus, the difference between unique sums of 40 and 70 may be much less significant than the difference between unique sums of 90 and 100. This does not represent a serious drawback as long as only rank ordering of alternatives is involved. If it is necessary to know to what degree the condition of one section differs from that of another, this disadvantage could be very significant. For many network level applications, the former seems to be the case, so that the unique sum technique offers considerable promise for network level pavement management applications.

TABLE 39 UNIQUE SUM CATEGORIZATION FOR THREE ATTRIBUTES WITH FOUR LEVELS (AFTER REF. 72)

Attribute	Level	Value
A	1	12
	2	18
	3	48
	4	80
B	1	17
	2	24
	3	55
	4	93
C	1	1
	2	5
	3	15
	4	20

TABLE 40 UNIQUE SUM CATEGORIZATION FOR FOUR ATTRIBUTE WITH FOUR LEVELS

Attribute	Level	Value
A	1	5
	2	9
	3	13
	4	17
B	1	1
	2	2
	3	3
	4	4
C	1	18
	2	35
	3	52
	4	69
D	1	70
	2	139
	3	208
	4	277

CHAPTER 7. RECOMMENDATIONS

The concept of pavement serviceability-performance has been widely accepted since the AASHO Road Test and its development by Carey and Irick (Ref. 1). The results of the AASHO Road Test gave an extremely useful empirical relationship for relating pavement design material properties and structural thicknesses to pavement performance. In trying to apply these results in a much wider variety of circumstances, significant problems have arisen with regard to the limited basis for the empirical equation. For example, only one subgrade was used and only one regional or environmental factor is involved. Nevertheless, the AASHO Road Test results have been widely used for the past twenty years in the form of the AASHTO Interim Pavement Design Guide (Ref. 73)[1] and in many other ways.

SIGNIFICANT PROBLEMS ENCOUNTERED

In recent years a strenuous effort has been made to model pavement behavior with theoretical or more widely applicable models. As outlined in this report, these include fatigue models of stress and strain, models of visco-elastic behavior, as well as models of low temperature and other environmental behavior. The critical step in the application of any model, however, is the verification of the model with experimental results. There have been no major new experimental results developed in the pavement behavior-serviceability-performance area since the AASHO Road Test. The extensive studies described in this report have pointed out the inadequacy of existing field pavement observation.

The importance of relating pavement distress to a performance or failure function was well defined in the special TRB conference on structural design of asphalt concrete pavement systems (Ref. 74)[2]. In summarizing research needs, the advisory committee pointed out that the mechanistic approach to pavement design, while it is vital to extensions of empirical data, can at best yield predictions of the nature and extent of pavement distress (e.g., the extent of rutting, cracking, faulting, spalling, etc.). The committee pointed out the urgent need for relating distress to the functional performance of the pavement and if possible the need to develop an ultimate failure prediction model.

It was recommended that the only feasible way to relate distress to

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[1]AASHTO Interim Guide for Design of Pavement Structures, 1972, American Association of State Highway and Transportation Officials, 1974.

[2]Special Report 126, Transportation Research Board, Structural Design of Asphalt Concrete Pavement Systems, Washington, D.C., 1971.

performance was through a statistical analysis of serviceability-performance information and objective distress predictions or evaluations. The committee suggested that such an analysis was required to (a) define important distress factors involved in pavement nonserviceability and failure, (b) establish suitable weighting functions to judge the relative importance of the various levels of combined distress modes, (c) identify suitable limiting levels of distress occurring separately or in combination, and (d) develop or adapt suitable measures of performance or serviceability.

In his excellent paper in Reference 74, Dr. Karl Pister points out that "...in the present milieu of rapid change, experience quickly becomes obsolete or is often totally lacking...thus it is important to have a 'theory' of pavement design". Dr. Pister also pointed out that "it seems to me that the existence of a rational method of design has to be established a posteriori; i.e., it is the task of the engineer to observe, acquire and organize information and experience obtained from operational physical systems".

It was also clearly pointed out in the summary of research needs by the advisory committee to the conference (Ref. 74) that the design, behavior and performance of pavement structures was basically a stochastic process, because the pavement system response reflects the statistical variations in the input variables (such as load, environment, pavement geometry, and material properties, including the effects of construction and testing variability).

Pavement Condition Variables

These two major areas of concern treated in 1970 reflect directly the findings of our current project. Strenuous effort has been made in this research to seek out the available data on pavement distress and performance in a sincere attempt to relate these factors for a variety of real world conditions. As pointed out in the results, the available data bases are inadequate for this purpose for several reasons. In many cases it is clear that the complexity of the models for predicting distress require significant input parameters which have not been collected as part of previously planned pavement information data banks. This should not be surprising to practicing engineers, although there seems to be a strong tendency to expect that currently available data would be appropriate to develop and verify models for relating distress to performance.

This situation is obviously aggravated by the second major factor discussed above: the natural statistical variation present in the pavement system. As pointed out in this report, these statistical variations undoubtedly greatly affect the data which have been made available to us. In the face of the large amount of statistical variation observable in the field it has not been possible to predict direct pavement distress-performance relationships.

It is clear from the work in this project that the required distress-performance relationships can come in the scientific way from one of two approaches:

(a) Improve the quality of individual observations so that a minimum number of observations accurately taken will provide statistically and mechanistically significant results.

(b) Take a significant and sufficient amount of data such that the overall variability is overcome by the quantity of data and that statistically significant results can be developed.

Finally, it appears that it will be necessary that a more precise statement about data required, sampling and measurement procedures, and the extent and time base of the relationships must be established explicitly if meaningful results are to be obtained. Too often in the past, data taken for one specific purpose have been expected at a later date to fulfill other widely different needs. While in some cases this approach works satisfactorily, in the case of pavement observations there is little evidence to suggest potential success in this approach.

Time Effects

The other major problem associated with the development of meaningful distress-performance relationships is the time factor. While at the AASHO Road Test a total of more than 1,100,000 heavy axle loads were applied on the various pavement loops, only two annual environmental cycles were suffered by these pavements. The interaction of environment and load appears to be much more significant than can be accounted for at the AASHO Road Test. Furthermore, the form of the serviceability history models appears to indicate that the models themselves are very nonlinear and that break points or trends of the curves cannot be easily established with a short time frame. This effect is aggravated by the variability of the problem. For example, a large portion of the serviceability-performance data reported and examined herein exhibit trends showing improving serviceability for a portion of the pavement history, in some cases over time frames of five or six years. This time effect factor can only be treated with the continued and conscientious observation of field serviceability and performance for periods of ten or more years; however, considerable insight into these trends can be gained through accelerated pavement testing. The Federal Highway Administration currently has under development an experimental accelerated pavement testing facility called APTS. Properly combined with field observations of pavements such that long term time and environmental effects can properly be accounted for, such an accelerated testing program may prove to be beneficial.

Serviceability or Roughness Measurements and Calibrations

A final factor affecting the problem of relating distress to perform-

ance is the quality of serviceability or roughness measurements. In any relationship the precision and accuracy of the factors being related on both sides of the equation are vitally important. In the case of relating distress to serviceability-performance a major problem exists with the time stability of roughness measurements which are used in most cases to define serviceability-performance.

Significant studies by Hudson, Haas, Roberts and others (Refs. 75[1], 76[2], 77[3], 78[4], 79[5]), have pointed out the difficulty in maintaining a stable calibration of pavement roughness measuring devices. The many factors related to the reaction of a pavement roughness device include all the dynamic aspects of the measuring vehicle as well as the roughness characteristics of the road surface. It is essential to maintain a stable roughness time base over a time frame of at least ten years, otherwise any variability in the roughness device is expressed as variation in the serviceability of the pavements being observed. In the data taken to date this variability of roughness measurement has caused significant loss of predicting capability in the overall equations. While the technology exists to solve this measurement problem, the costs appear to be relatively high. In the transportation field, where, traditionally, relatively inexpensive equipment has been utilized, the seemingly high cost of calibrating roughness equipment is not well received. Nevertheless, in order to solve this problem, calibratable roughness equipment is essential. There does exist a significant piece of equipment, the General Motors Surface Dynamics Road Profilometer, which can be used both to measure highway roughness and serviceability,

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[1]Roberts, F.L. and W.R. Hudson, "Pavement Serviceability Equations Using the Surface Dynamics Profilometer", Research Report 73-3, Center for Highway Research, University of Texas, 1970.

[2]Haas, R., "Surface Evaluation of Pavements: State-of-the-Art", A paper contained in proceedings of San Francisco Workshop on Pavement Rehabilitation, Report No. DOT-OS-40022, 1974.

[3]Walker, R.S., W.R. Hudson, and F.L. Roberts, "Development of a System for High Speed Measurement of Pavement Roughness, Final Report", Research Report 73-5F, Center for Highway Research, University of Texas, 1971.

[4]Roberts, F.L., "The State-of-the-Art of Estimating Pavement Serviceability Using Roughness Measurements", Proceedings of an A.S.C.E. Specialty Conference on Pavement Designs for Practicing Engineers, June 1975.

[5]Special Report 116, Highway Research Board, Improving Pavement and Bridge Deck Performance, Washington, D.C., 1971.

but more importantly, to calibrate other less expensive instruments. The cost of such quality roughness measurements is small when counted against the benefits of providing an adequate data base of pavement serviceability and performance history for improved pavement models and thus improved overall pavement quality.

Overall Performance Concepts

The Carey-Irick definition of pavement performance is used throughout this report and has come to be accepted by most groups as the standard for pavement performance discussions. No attempt has been made in this study to change this definition since the problem statement and the problem of relating distress to performance has been well defined since the FHWA workshop in Austin, Texas, in 1970 (Ref. 74) and in subsequent Transportation Research Board task force activities. However, research since 1973 and the data collected and described in this report show an additional need to define pavement performance more broadly. No specific titles have been put forth but the concept of "Overall Performance" must be considered in future research. Such an "Overall Performance" function or OP Function could define in a more complete way the various output parameters which are important in defining the system output function of pavement management. In such an OP Function, consideration would be given to riding quality (the factor most completely evaluated by serviceability), safety or skid resistance, pavement condition, maintenance cost, and other factors.

The need for this "Overall Performance" function is highlighted by the inability of the serviceability-performance concept to account adequately for safety due to skid resistance, or excessive rut depths. This need is also emphasized by the number of cases observed in practice where the pavement serviceability history does not appear to deteriorate with time in the same fashion that was observed at the AASHO Road Test. The deterioration rate rather seems to be heavily dependent upon the maintenance budget and the maintenance characteristics performed upon the pavement sections under observation. Careful consideration indicates this may not be an unexpected situation since little or no maintenance was performed during the two years of the AASHO Road Test on the pavement sections and yet generally good maintenance is performed, for example, on the Interstate Highway System.

This concern for an "Overall Performance" function is also highlighted by the fact that most highway agencies feel that heavy duty highways and freeways such as the Interstate Highway System and particularly the Urban Interstate System cannot be allowed to deteriorate below serviceability levels of 3.0 or even 3.5 in some opinions.

As stated, no specific treatment is made of this fact within this study, however, the authors feel strongly that the matter should be considered in additional studies subsequently.

RECOMMENDATIONS

For the past several years a significant trend has been evident which can provide an important solution to the problem at hand. This involves the implementation of pavement management systems in a number of state highway departments. A part of any such pavement management system is a significant, long-term data base of pavement condition values and performance observations. These observations are made for the explicit reason of relating pavement serviceability-performance to the pavement management models and cost functions. Thus, there is high probability that the data wil be taken for a long period of time, and taken in a more appropriate manner than may have previously been the case. It is essential, however, that close attention be given to the calibration of such observations over a long period of time if the problem is to be solved. The more extensive use of the Surface Dynamic Road Profilometer for calibration is encouraged, since no other proven technology for calibrated roughness measurements exists at the present time. Experience has shown that a period of at least 3-5 years is required for a new technology to be proven and put into service.

The combination of these high quality long-term observations of pavement serviceability performance along with the appropriate input variables and pavement distress observations can be used in conjunction with accelerated pavement testing to be carried out by the Federal Highway Administration to provide the missing link of distress-performance relationships which is so vital to closing the loop in pavement modeling and management equations.

With regard to these important areas of concern, a series of research problem statements are outlined below.

Research Problem Statement 1. Pavement Performance Models Using Combined Data from Accelerated Pavement Testing and Long-Term Field Performance Observations

Conscientious observations of existing pavement performance are essential in order to develop reliable pavement performance relationships for a variety of environmental traffic and material conditions over the entire United States, and to relate pavement performance to significant factors such as pavement distress, environment, traffic loading, and maintenance practices. The Federal Highway Administration is developing an accelerated pavement testing system which potentially can be used to develop data related to the performance of new pavement materials and design systems. It is proposed that a combined approach be carried out, beginning as soon as possible, to collect significant information on the performance

of pavement materials under a variety of traffic and environmental conditions, and that this include observations of pavement performance in the field under existing conditons and under accelerated testing conditions.

Field conditions should include observations of already existing pavement sections as well as any experimental or special sections which are constructed as a result of innovative materials, construction or design techniques. Thus, in the final analysis it will be essential to combine: a) the results of accelerated testing procedures, b) the results of long term observations of existing pavements, and, c) the specialized field observation of test sections or experimental sections to develop the overall methodologies and models which will be needed for the management of the nation's pavements in the future.

This research project is extremely urgent since the absence of rational pavement performance models is hampering the developing of improved pavement design and performance data throughout the United States at the present time. It is thus urged that conscientious field observations begin at the earliest possible date in as many states as possible and that the accelerated pavement testing program of the Federal Highway Administration be coordinated with this data collection operation.

Also, it is extremely important that the experimental testing in the accelerated pavement program be carried out through a well-designed experiment in order to provide statistically complete data that can be used for future performance predictions. Significant resources and time can be wasted unless satisfactory statistical relationships are employed in the accelerated pavement experiment design.

Research Problem Statement 2. Long Term Pavement Observations for Pavement Management Systems

As outlined above, significant strides have been made in the collection of pavement serviceability and distress data as part of pavement management systems being implemented in a number of state departments of transportation. It's essential that closer attention be given to this important aspect of the pavement management system and that essential guidelines for data to be taken, sampling frequencies, time histories of sampling, and quality of data measurement, be provided to ensure that the data being taken will be valuable ten years hence. The adequacy and accuracy of such data are important stumbling blocks in the implementation of pavement management systems. Each state can, of course, develop an independent pavement management system. However, in order to provide the greatest potential for the use of models based on accelerated pavement testing and field observation throughout the nation, some standardization of pavement distress, maintenance and performance measures is desirable. Attention now to the use of standard units for measuring pavement condition variables and standard procedures for roughness

measurement or serviceability rating by panels can assure that long term data records kept by the individual states are mutually compatible and complimentary. This will greatly increase the potential for effective performance model development and verification, to the benefit of all concerned.

Problem Statement 3. Improved Serviceability Measurements and Roughness Calibration Technique

It's essential that pavement roughness measuring techniques and serviceability-performance prediction techniques be improved and applied as part of the long term observations of pavement serviceability. Significant research in Texas, at the Center for Highway Research (Refs. 75, 77, 80[1], 81[2]), have clearly shown that the Surface Dynamic Profilometer can be used as a high quality pavement serviceability measuring device, and that it can also serve as a calibrating device for less costly roughness measuring equipment such as the PCA meter and the Mays Meter. This technique has also been used on a multi-million dollar pavement research project carried out in Brazil (Refs. 82[3], 83[4]). If the problem of relating distress to performance is to be solved, it is essential that known technology in pavement roughness measurement be implemented and that additional research be initiated to continue development of non-contact probes and other high quality roughness measuring equipment.

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[1]Williamson, H.J. and W.R. Hudson, "Analysis of Characteristic Roughness Patterns in Pavements and the Relationship Between Roughness and Pavement Distress", Center for Highway Research, Research Report 156-3, The University of Texas at Austin, 1974.

[2]Smith, R.P. and B.F. McCullough, "The Use of Condition Surveys, Profile Studies and Maintenance Studies in Relating Pavement Distress to Pavement Performance", Research Report 123-19, Center for Highway Research, The University of Texas at Austin, 1974.

[3]Empresa Brasileira de Planejamento de Transportes - GEIPOT - Report I, Inception Report, Research Concepts and Procedures, Research on the Interrelationships between Costs of Highway Construction, Maintenance and Utilization - GEIPOT, Brasilia, Brazil, May 1976.

[4]Empresa Brasileira de Planejamento de Transportes - GEIPOT - Report II, Midterm Report, Preliminary Results and Analyses, Research on the Interrelationships between Costs of Highway Construction, Maintenance and Utilization - GEIPOT, Brasilia, Brazil, August 1977.

A possible alternative approach to improved serviceability measurement would involve development of more reliable and efficient panel rating procedures. New York has taken the lead in this area, and has developed a procedure which offers a definite improvement over traditional techniques (Refs. 44, 84[1], 85). The widespread implementation of reliable, standard panel rating techniques could provide a supplement or substitute for improved roughness measurement procedures.

Problem Statement 4. Coordination of Activities Over a Range of Environment, Load, Materials and Other Factors

As pointed out in this report there is a large variability in many of the factors associated with pavement serviceability and distress observations. In order to overcome this Irick and Hudson in NCHRP Reports 2 and 2a (Refs. 86[2], 87[3]), pointed out the desirability of careful observations of selected pavement sections on the highway network. These are either to be specially constructed or carefully selected from existing pavements to yield high quality data. Furthermore, it was suggested that these pavements be selected over a wide range of conditions. For example, to investigate environment, pavements would be selected in Florida as one extreme and Minnesota as another extreme. Louisiana and Arizona, for example, could serve as extremes for rainfall. By observing factors and results over extreme ranges of parameters, it is statistically much easier to define significant results and thus to build useful models. Thus, there is significant benefit in coordination of a program of pavement observation among many of the states under the auspices of the Federal Highway Administration or the National Cooperative Highway Research Program. If such observations are coupled with the implementation of pavement management systems, then the benefits and value of cooperation become even more apparent since improved pavement management and data bases are important to all of the states involved.

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[1]Weaver, R.J., "Quantifying Pavement Serviceability as Judged by Highway Users", prepared for presentation at the 58th Annual Meeting of the Transportation Research Board, Washington, D.C., January 1979.

[2]Irick, P.E., "An Introduction to Guidelines for Satellite Studies of Pavement Performance", NCHRP Report 2, Highway Research Board, 1964.

[3]Irick, P.E. and W.R. Hudson, "Guidelines for Satellite Studies of Pavement Performance", NCHRP Report 2A, Highway Research Board, 1964.

SUMMARY

The pavement distress/performance problem is quite complex, and has clearly not been fully resolved by this research effort. Nevertheless, some progress in this important area has been made. Specific distress types believed to have the most significant influence on pavement performance have been selected, typical distress and serviceability history patterns have been identified, and existing mechanistic models for distress prediction have been evaluated. A theoretical basis for the development for pavement distress has been formulated for each selected important distress type. Several time-independent and time-dependent distress/ serviceability equations have been developed on the basis of data collected from state transportation agencies, special research studies and published reports, as well as subjective data.

Some significant problems were encountered, especially with regard to the adequacy of the available data. The development of good models requires carefully recorded data of a type and extent not generally required of data collected for routine monitoring or other purposes. Hence, data from special research studies were found to be most useful in this project. Specific data requirements for performance modeling have been set forth herein, along with a designed experiment approach for obtaining such data.

The primary applications of this research will most likely involve pavement management. Consequently, performance model requirements for pavement management have been discussed, along with the need for an overall measure of pavement performance. Two methods for achieving such an overall performance variable are presented, and examples of each are provided.

Because of the problems discussed herein, attempts to develop equations expressing direct relationships between distress and serviceability met with limited success. The regression equations presented in this report are therefore not considered suitable for immediate use by state agencies in pavement management. The findings of this research indicate that agencies desiring to use direct modeling of distress/performance relationships should consider the possibility of using separate equations for each pavement section.

Finally, several problem statements have been provided to address key areas for future research in this field. The major thrust of these problem statements is directed toward the acquisition of better data bases, both for modeling and management purposes. An indirect approach to the use of distress/performance relationships in performance modeling for pavement management has also been suggested. An example of the application of Markovian techniques in this approach is provided. The primary requirement for such modeling, however, is that recursive or

self-modifying methods be involved; the specific mathematical formalism to be used will depend upon the particular application. Of course, more direct modeling based upon reliable objective data is to be desired. However, since the collection of quality research data, as outlined in this chapter, will require an appreciable amount of time, it is likely that performance predictions based on recursive or self-modifying techniques will offer the greatest promise for reliable performance prediction for the next several years.

APPENDIX A

DEFINITION OF TERMS

The following definitions of terms relating to pavement distress and performance were selected as appropriate for this study. These definitions follow those used in several previous investigations (Refs. 18, 19, 20).

1. Distress is a condition of a pavement structure which reduces serviceability or leads to a reduction of serviceability.

2. Distress Manifestations are the visible consequences of various distress mechanisms, which usually lead to a reduction in serviceability.

3. A Distress Mechanism is the physical or chemical process involved in or responsible for distress in pavements.

4. Response is the reaction of a pavement structure to load and environment.

5. Primary Responses are those responses which, when carried past some limiting value, initiate distress.

6. Other Responses are those responses which do not contribute directly to distress.

7. A Response Mechanism is the physical or chemical process responsible for the response of a pavement structure.

8. Serviceability is the ability of a specific section of pavement to serve traffic in its existing condition.

9. Serviceability Index is a numerical estimate of serviceability based on pavement roughness.

10. Serviceability History is a representation of the serviceability of a pavement over a period of time. Serviceability history is sometimes termed "performance", and is generally given by serviceability index as a function of time or accumulated load.

11. Performance is a measure of accumulated service provided by a facility; i.e., the degree to which a pavement fulfills its purpose.

12. Fracture is the state of a pavement material being broken.

13. <u>Distortion</u> is a permanent change in the shape of the pavement or pavement component.

14. <u>Disintegration</u> is the state of being decomposed or abraded into constitutive elements.

15. <u>Bleeding</u> is the condition of free bitumen on the surface of the pavement due to excessive bitumen and/or insufficient void space.

16. <u>Reflection Cracks</u> are cracks occurring in the surface course of a pavement that coincide with and are caused by the relative movement of cracks or joints in underlying layers.

17. <u>Low-Temperature Cracks</u> are cracks (generally transverse) caused when tensile stresses induced by frictional resistance of the underlying layer to thermal contraction of the surface layer exceeds the tensile strength of the surface material.

18. <u>Raveling</u> is the progressive disintegration of an asphalt concrete layer from the surface downward by the dislodgement of aggregate particles. This may be caused by insufficient amount of binder in the mix, hardening of the asphalt binder, wet or dirty aggregate or aggregate with smooth surface texture.

19. <u>Ruts</u> are longitudinal depressions that form in the wheel paths of flexible or composite pavements, resulting from compaction or lateral migration of one or more of the pavement layer materials under the action of traffic and environment.

20. <u>Reduced Skid Resistance</u> is the reduction of frictional resistance between tires and a pavement surface. This reduction is generally due to abrasive wear of aggregates by traffic.

21. <u>Shrinkage Cracks</u> are generally transverse cracks caused when tensile stresses induced by frictional resistance of the underlying layer to drying contraction of the surface layer exceeds the tensile strength of the surface material. These cracks generally occur in portland cement concrete and other cement treated materials.

22. <u>Spalling</u> is cracking, breaking, or chipping of a rigid pavement along joints, edges, or cracks in which small portions of the slab are dislodged.

23. <u>Faulting</u> is a difference in the elevation of two adjacent rigid slabs at the joint or crack interface due to consolidation or swelling of underlying material, inadequate load transfer, or pumping.

24. "D" Cracking is a series of fine, crescent-shaped hairline cracks in a rigid slab surface, usually paralleling a joint or major crack.

25. Steel Rupture is the occurrence of a tensile fracture failure in the reinforcing steel when excessive stress is transferred upon fracture of adjacent concrete.

26. Polished Aggregates are surface aggregate particles having smooth, rounded surfaces with fine microtexture, either as original condition or after abrasive wear by traffic.

27. Punchouts are blocks of rigid pavement that are cracked around their periphery and displaced downward relative to the rest of the slab. Punchouts usually occur between closely-spaced transverse cracks that are subsequently connected by longitudinal cracks.

28. Fatigue Cracks are cracks in a pavement layer caused by the combination of repetitive strains and apparent reduction of tensile strength due to fatiguing of the layer material. The repetitive strains causing fatigue are usually caused by passing wheel loads, but may include repetitive thermally-induced or other strains.

APPENDIX B

SELECTION OF PAVEMENT DISTRESS TYPES FOR FURTHER STUDY

Tables 41 through 53 give distress manifestations from selected sources. Tables 41 and 42 summarize the comprehensive distress survey of Barenberg, et al (Ref. 21). This particular study is directed toward recognition and repair of pavement distress, so that the distress types are categorized by distress manifestation rather than by mechanism. In addition, this study involves 0-70 mph (0-113 kph), 0-40 mph (0-64 kph), and 0-20 mph (0-32 kph) roads (as well as parking lots), urban and rural, primary and secondary roads, and hence, includes some distresses that are not considered applicable to this project. Nevertheless, it is a very complete survey which was considered to provide a complete catalog of pavement distress by type, severity, and cause.

Darter and Barenberg (Ref. 22) surveyed distress occurrences in the field and ranked the types of distress in order of frequency or occurrence. These results are listed in Tables 43 through 47. This study was based on distress measurements in several states, and only premium pavements were considered. Thus, the distress types listed here are more nearly in line with the requirements of the present work, and in general, the distress types near the top of their rankings agree roughly with those to be used in this project. However, frequency of occurrence may not reflect the impact of a type of distress on serviceability/performance, so the order cannot be adopted directly. In addition, the rankings of Reference 22 are based on distress manifestations in pavements which survived for 20 years, a restriction which is not relevant to the present work.

A third report which offers insight into the relative importance of various types of distress was recently completed by researchers at the Texas Transportation Institute (Ref. 23). This report gives "deduct values" for different degrees of severity for various types of distress. Tables 48 and 49 list these deduct values for flexible and rigid pavements, respectively.

These deduct values are intended for use in calculating a pavement rating score (PRS) defined by the following equation:

$$PRS = C - \sum_{i=1}^{n} \sum_{j=1}^{n} a\,(T_i, S_j, E_{ij})$$

where:

C = initial rating score of the pavement, and
$a(...)$ = Weighting factor (deduct value) dependent upon distress Type T_i, Severity S_j, and the extent of distress E_{ij}.

TABLE 41 **FLEXIBLE PAVEMENT DISTRESS (REFERENCE 21)**

	Distress	Severity	Cause
1.	Abrasion	high	hard material scraping on the surface of the road.
2.	Bleeding	very high	excessive bitumen, or excessive compaction to reduce air voids
3.	Char	very low	fire burning on pavement surface.
4.	Indentation	very low	punching shear forces due to sharp or heavy metal objects.
5.	Loss of cover aggregate	very high	inadequate binding of aggregate in surface treatment.
6.	Polished aggregate	very high	poor abrasion resistance of aggregate.
7.	Pothole	very high	poor surface mixture allows traffic to abrade pavement from weak spots.
8.	Raveling	high	inadequate binding of aggregate due to dirty aggregate, improper mix or improper compaction.
9.	Streaking	very high	improper asphalt distribution during surface treatment.
10.	Weathering	moderate	binder becomes brittle and less adhesive with time, allowing surface abrasion.
11.	Corrugation	very high	unstable mix due to inadequate binder, aggregate or curing.
12.	Cracking, Alligator	high	excessive deflection of stiff mix over a relatively resilient base.
13.	Cracking, Contraction	moderate	stiff binder or improper bonding to underlying layers cannot accommodate thermal volume change.
14.	Cracking, Edge	high	insufficient vertical and/or lateral support at shoulder.
15.	Cracking, Edge Joint	high	separation of pavement and shoulder due to frost heave, subgrade volume change or heavy wheel loads.
16.	Cracking, Lane Joint	moderate	improper (cold) bonding of adjacent lanes of pavement.
17.	Cracking, Reflection	moderate	vertical and/or horizontal movement along existing joints in underlying layers.
18.	Cracking, Shrinkage	moderate	excessive volume change due to high binder content or high coefficient of thermal expansion, low penetration, or weathered asphalt.
19.	Cracking, Slippage	high	poor bond of surface to underlying layers or unstable surface mixture.

TABLE 41 FLEXIBLE PAVEMENT DISTRESS (REFERENCE 21) (Continued)

	Distress	Severity	Cause
20.	Depression	high	traffic-related compaction of surface or base, or settlement of fill material.
21.	Rutting	high	traffic-related compaction or shaving due to improper compaction or unstable mix.
22.	Shoving	very high	unstable pavement surface or base due to excess or poor binder, poor aggregate or excess moisture.
23.	Upheaval	very high	local expansion in subgrade or pavement layer due to frost or soil expansion.
24.	Utility Cut Depression	very high	improper backfill following utility construction.

TABLE 42 RIGID PAVEMENT DISTRESS (REFERENCE 21)

	Name	Severity	Cause
1.	Crazing	low to moderate	excessive finishing of slab surface.
2.	Joint Filler Extrusion and/or Stripping	moderate to high	poor adhesion between joint and filler, improper application or excessive joint filler.
3.	Scaling	moderate	de-icing chemicals; improper mixing, curling or finishing; or unsuitable aggregates.
4.	Buckling Blow-up	very high	insufficient joint width or foreign material in joints.
5.	Shattering Blow-up	very high	same as buckling blow-up.
6.	Corner Cracking	high	poor support on slab corners due to pumping, curling or soft base.
7.	Diagonal Cracking	moderate to high	same as corner cracking on a larger scale.
8.	Longitudinal Cracking	high	lateral shrinkage with inadequate joints, loss of edge support, or expansive subgrade soils.
9.	Transverse Cracking	high to very high	slab contraction with inadequate joints, curling and pumping.
10.	Second Stage Cracking	high to very high	loss of slab end support from pumping, improper load transfer at joints.
11.	Progressive Cracking	very high	heavy loads on already cracked, poorly supported pavement.
12.	Random Cracking	high	extreme overloading and lack of adequate roadbeα support.
13.	Curling	moderate to high	non-uniform slab expansion/contraction caused by heat or moisture differences from top to bottom.
14.	"D" Cracking	moderate	structural breakdown due to freeze/thaw and variably expansion aggregate.
15.	Faulting	high to very high	differential consolidation or swelling of underlying material, or pumping, coupled with inadequate load transfer.
16.	Joint Failure	high	impressibles in joint, or compressive bending stresses.

TABLE 42 RIGID PAVEMENT DISTRESS (REFERENCE 21) (Continued)

	Name	Severity	Cause
17.	Pumping	high to very high	free water in base, subbase or subgrade and insufficient slab support.
18.	Spalling	high	incompressibles in joint, improper joint construction or load transfer, inferior concrete or pavement too thin.

TABLE 43 SUMMARY OF DISTRESS TYPES FOUND ON FLEXIBLE PAVEMENTS RATED AS MODERATE TO SEVERE (REFERENCE 22)

Types of Distress	PROJECTS Distresses/Total	Maintained/Distressed
1. Longitudinal Cracking (lane joint in nearly all cases)	11/19	** 5/11
2. Transverse Cracking (including reflective)	10/19	7/10
3. Alligator (fatigue) cracking	9/19	5/9
4. Polished Aggregate	8/19	0/8
5. Rutting	6/19	1/6
6. Weathering Asphalt	4/19	0/4
7. Depressions	3/19	0/3
8. Alligator or Transverse Cracking*	14/19	9/14
9. Alligator or Transverse or Longitudinal Cracking or Rutting*	17/19	10/17

*Whichever of the distress types was rated highest for each pavement.
**Maintenance performed only for distress indicated.

TABLE 44 SUMMARY OF DISTRESS TYPES ON COMPOSITE PAVEMENTS WHERE DISTRESS IS RATED AS MODERATE TO SEVERE (REFERENCE 22)

Types of Distress	PROJECTS Distresses/Total	Maintained*/Distressed
1. Transverse Cracking	6/7	2/6
2. Rutting	5/7	0/5
3. Longitudinal Cracking	4/7	2/4
4. Edge Cracking	3/7	1/3
5.. Random Cracking	3/7	1/3
6. Depression	2/7	0/2
7. Polished Aggregate	2/7	0/2
8. Raveling and Weathering	1/7	0/1

*Maintenance performed for only the distress indicated.

TABLE 45 SUMMARY OF DISTRESS TYPES ON JCP RATED AS MODERATE TO SEVERE (REFERENCE 22)

Types of Distress	PROJECTS Distresses/Total	Maintained*/Distressed
1. Joint Filler Extrusion and/or Stripping	11/11	2/11
2. Shoulder Distress	4/8	1/4
3. Faulting Joints	4/12	2/4
4. "D" Cracking	3/12	1/3
5. Interconnecting Cracking	2/12	0/2
6. Surface Depression	2/12	0/2
7. Transverse Cracking	2/12	2/2
8. Joint Spalling	1/12	0/1

*Maintenance applied to only specific distress indicated.

TABLE 46 SUMMARY OF DISTRESS TYPES FOUND ON JRCP RATED AS MODERATE TO SEVERE (REFERENCE 22)

Types of Distress	PROJECTS Distresses/Total	Maintained*/Distressed
1. Transverse Cracking	14/18	9/14
2. Paved Shoulder Distress	6/8	4/6
3. Faulting of Cracks	11/18	6/11
4. Joint Filler Stripping	11/18	5/11
5. Joint Spalling	9/18	7/9
6. Surface Depressions or Swell	4/18	4/4
7. Corner Cracking	6/18	5/6
8. Longitudinal Cracking	3/18	2/3
9. Blowups	4/18	4/4
10. Diagonal Cracking	8/18	1/8
11. "D" Cracking	2/18	2/2

*Maintenance applied to only specific distress indicated.

TABLE 47 SUMMARY OF THE MOST PREDOMINANT DISTRESS TYPES FOUND ON CRCP RATED MODERATE TO SEVERE (REFERENCE 22)

Types of Distress	PROJECTS Distresses/Total	Maintained*/Distressed
1. Surface Depression	7/12	0/7
2. Crack Spalling	6/12	2/6
3. Punchouts	4/12	4/4
4. Interconnecting Cracks	4/12	2/4
5. Longitudinal Cracking	2/12	0/2
6. Steel Rupture	2/12	2/2

*Maintenance applied to only specific distress indicated.

TABLE 48 DEDUCT VALUES FOR FLEXIBLE PAVEMENTS (REFERENCE 23)

Type of Distress		Severity of Distress	Deduct Values Percent of Pavement Area Affected		
			(1-15%)	(16-30%)	(>30%)
1.	Rutting	slight	0	2	5
		moderate	5	7	10
		severe	10	12	15
2.	Raveling	slight	5	8	10
		moderate	10	12	15
		severe	15	18	20
3.	Flushing	slight	5	8	10
		moderate	10	12	15
		severe	15	18	20
4.	Corrugations	slight	5	8	10
		moderate	10	12	15
		severe	15	18	20
			(1-5%)	(6-25%)	(>25%)
5.	Alligator Cracking	slight	5	10	15
		moderate	10	15	20
		severe	15	20	25
			(1-5%)	(6-15%)	(>16%)
6.	Patching	good	0	2	5
		fair	5	7	10
		poor	7	15	20
7.	Longitudinal Cracking		Lineal Feet per Station - Lane		
			(10-99)	(100-199)	(>200)
	Case I: Cracks Sealed	slight	2	5	8
		moderate	5	8	10
		severe	8	10	15
	Case II: Cracks Partially Sealed	slight	3	7	12
		moderate	7	12	15
		severe	12	15	20
	Case III:Cracks Not Sealed	slight	5	10	20
		moderate	10	15	20
		severe	15	20	25

1 foot = 0.305 m.

TABLE 48 DEDUCT VALUES FOR FLEXIBLE PAVEMENTS (REFERENCE 23) (Continued)

8. Traverse Cracking		Number Per Station - Lane		
		(1-4)	(5-9)	(>10)
Case I: Cracks Sealed	slight	2	5	8
	moderate	5	8	10
	severe	8	10	15
Case II: Cracks Partially Sealed	slight	3	7	10
	moderate	7	10	15
	severe	10	15	20
Case III: Cracks Not Sealed	slight	3	7	12
	moderate	7	12	15
	severe	12	15	20

	Failures/Mile		
	(1-5)	(6-10)	(>10)
9. Failures/Mile	20	30	40

(Note: All Failures are considered to have a "Severe" degree of distress.)

1 mile = 1.6 km.

TABLE 49 **DEDUCT VALUES FOR RIGID PAVEMENT (REFERENCE 23)**

Type of Distress	Severity of Distress	Deduct Values		
		Variation in Extent Not Applicable		
1. Pumping	slight		20	
	moderate		40	
	severe		60	
		Failures/Mile		
2. Failures/Mile		(1-5)	(6-10)	(>10)
Note: All failures are considered to have a "severe" degree of distress		20	30	40
		Percent of Pavement Area Affected		
		(1-15%)	(16-50%)	(>50%)
3. Surface Deterioration	slight	5	10	20
	moderate	10	20	30
	severe	20	40	60
		Percent of Joints or Cracks Affected		
		(1-15%)	(16-50%)	(>50%)
4. Spalling	slight	5	10	15
	moderate	10	15	20
	severe	20	40	60
		Lineal Feet Per Station - Lane		
		(0-99)	(100-199)	(>200)
5. Longitudinal Cracking	slight	5	10	15
	moderate	10	15	20
	severe	15	20	25
		Number Per Station - Lane		
		(<10)	(>10)	
6. Faulting	moderate	5	15	
	severe	15	40	
		Distance Between CRCP Cracks		
		(5-10 ft.)	(Other:<5 ft or >10 ft)	
7. Crack Spacing (CRCP Only)	closed	0	10	
	open	15	40	
		Percent of Cracks Which Intersect		
		(<10%)	(>10%)	
8. Y Cracking	moderate	5	15	
	severe	15	40	

TABLE 49 DEDUCT VALUES FOR RIGID PAVEMENT (REFERENCE 23) (Continued)

Type of Distress	Severity of Distress	Deduct Values		
		Number per panel		
9. Transverse Cracking		(1-2)	(3-4)	(>4)
Case I: If Joint spacing is less than 20 feet	slight	5	10	20
	moderate	10	20	30
	severe	15	30	40
Case II: If Joint spacing is greater than 20 feet	slight	0	5	10
	moderate	5	10	20
	severe	10	15	30

1 foot = 0.305 m.

1 mile = 1.6 km.

TABLE 50 WEIGHTS FOR FLEXIBLE PAVEMENT LEVEL OF SERVICE (REFERENCE 23)

Segment Rehabilitation

	Skid Number	Rutting	PSI	Cracking	PRS
Urban	1.0	0.9	0.8	0.8	0.5
Rural	0.5	0.6	0.7	1.0	0.8

Spot Rehabilitation

	Skid Number	Flushing	Failures/ mile	Alligator Cracking	Raveling	Corrugations
Urban	1.0	0.9	0.9	0.8	0.7	0.6
Rural	0.7	0.8	1.0	1.0	0.8	0.6

1 mile = 1.6 km.

TABLE 51 WEIGHTS FOR RIGID PAVEMENT LEVEL OF SERVICE (REFERENCE 23)

Segment Rehabilitation

	Skid Number	Cracking	Spalling	Surface Deterioration	Pumping	SPI	PRS
Urban	1.0	0.9	0.9	0.9	0.8	0.8	0.6
Rural	0.8	1.0	0.85	0.8	0.85	0.6	0.9

Spot Rehabilitation

	Skid Number	Failures/ Mile	Faulting	Spalling	Pumping	Surface Deterioration
Urban	1.0	0.9	0.9	0.9	0.85	0.8
Rural	0.7	1.0	0.9	0.8	0.9	0.8

1 mile = 1.6 km.

TABLE 52 PAVEMENT DISTRESS BY CATEGORY AND PAVEMENT TYPE

Pavement Type	Distress Category		
	Fracture	Distortion	Disintegration
Flexible Pavements	Fatigue Cracking Thermal Cracking Slippage Cracking*	Differential Frost Heave Differential Compaction-Swelling Shoving* Rutting Corrugations*	Stripping Raveling Reduced Skid Resistance
Rigid Pavements	Fatigue Cracking Shrinkage Cracking Thermal Cracking Blowups Spalling	Faulting Differential Frost Heave Differential Compaction-Swell	D-Cracking Scaling Reduced Skid Resistance
Composite Pavements	Fatigue Cracking Thermal Cracking Slippage Cracking* Reflection Cracking	Differential Frost Heave Differential Compaction-Swell Shoving* Rutting Corrugations*	Stripping Raveling Reduced Skid Resistance

*Not important in premium pavements using current design and construction techniques, but should be considered to ensure that they do not develop.

TABLE 53 PRIORITY RANKING OF SIGNIFICANT DISTRESS SELECTED FOR FURTHER STUDY (Reference 23)

Priority Ranking	Flexible	Rigid Pavements: JCP	Rigid Pavements: JRCP	Rigid Pavements: CRCP	Composite Pavements
1	Fatigue Cracking	Fatigue Cracking	Low-Temp. and Shrinkage Cracking	Crack Spalling	Reflection Cracking
2	Rutting	Joint Faulting	Fatigue Cracking	Fatigue Cracking	Fatigue Cracking
3	Low-Temp. Cracking	D-Cracking	Crack Faulting	Low-Temp. Cracking	Rutting
4	Reduced Skid Resistance	Joint Spalling	Joint Spalling	Shrinkage Cracking	Reduced Skid Resistance
5			D-Cracking	Punchouts	
6				Steel Rupture	

The higher the deduct value the more important the impact of that particular distress on the pavement rating. According to this criterion for flexible pavements, the most significant distresses are failures/mile (potholes), alligator cracking and longitudinal cracking. For rigid pavements the corresponding order is pumping, surface deterioration, and spalling.

In this same study it was recognized that the deduct values applied only to the immediate effects of a particular distress, whereas maintenance personnel would be influenced by projected future impacts, societal values, environmental considerations, etc., in deciding priorities for maintenance activities. Consequently, a series of interviews was conducted and a set of utilities was developed to reflect the relative importance of various distress, safety, and user comfort variables under different circumstances. These utilities are reproduced in Tables 50 and 51. The categories which correspond roughly to the needs of this project are "rural" and "segment rehabilitation", although some consideration has been given to the urban setting as well. This combination of utilities and deduct values is adequate because it is concise and yet flexible enough to apply to a wide variety of situations.

Finally, Tables 52 and 53 contain the distress categories of Rauhut et al (Ref. 18). This report is quite comprehensive: the various distresses are defined, the causative mechanisms are identified, important material properties are noted, and an attempt is made to demonstrate the complex interrelationships between primary and secondary distress manifestations. The focus here is on the causes and development of distress symptoms, so that the distress types are classified by cause rather than by field appearance. Hence, instead of longitudinal, transverse and alligator cracking, one finds fatigue, thermal and shrinkage cracking. This is much more desirable for theoretical analysis and model building, although correlation with field observation is more difficult.

The priority ranking of Table 53 (Ref. 18) is expected to be valid for this project with only minor modification. However, as an independent check, Reference 14 may be used to make an evaluation of the relative impact of various distress types. The combination of deduct values and utilities for each distress allows an assessment of both the immediate and expected future effects on serviceability/performance. This combination is accomplished as follows:

(1) Choose a set of utilities for the problem at hand. This project is concerned with both flexible and rigid pavements, a primarily rural setting, and primarily with rehabilitation of an entire segment of roadway. The utilities in Tables 50 and 51 were, therefore, combined in such a way as to emphasize the "rural" and "segment" values without ignoring the "urban" and "spot" contributions. This was done by defining new utilities,

UF_i = new utility of flexible pavements for distress type i,

UR_i = new utility for rigid pavements for distress type i,

in terms of the values quoted in Tables 50 and 51:

$$UF_i = \alpha_1[\alpha_2 UFSGR_i + (1-\alpha_2)\ UFSGU_i] + (1-\alpha_1)\ [\alpha_2 UFSPR_i + (1-\alpha_2) UFSPU_i]$$

$$UR_i = \alpha_1[\alpha_2 URSGR_i + (1-\alpha_2)\ URSGU_i] + (1-\alpha_1)\ [\alpha_2 URSPR_i + (1-\alpha_2) URSPU_i]$$

where

α_1 = fraction representing research interest in rural rather than urban pavements.

α_2 = Fraction representing research interest in segment rehabilitation rather than spot rehabilitation.

$UFSGR_1$ = flexible segment rural utility (from Table 50) for distress type i.

$UFSGU_1$ = flexible segment urban utility (from Table 50) for distress type i.

$UFSPR_1$ = flexible spot rural utility for type i distress.

$URSGU_i$ = rigid segment urban utility (from Table 51) for distress type i, etc.

$URSPU_i$ = rigid spot urban utility for distress type i.

$UFSPU_i$ = flexible spot urban utility for distress type i.

$URSGR_i$ = rigid segment rural utility for distress type i.

$URSPR_i$ = rigid spot rural utility for distress type i.

The numerical values for UF_i and UR_i are presented in Tables 54 and 55 for various combinations of α_1^i and α_2^i (α_1, α_2 = 2/3, 3/4, 4/5). The order of importance established by these utilities is nearly independent of the particular combination of α_1 and α_2, as can be seen from the values in the tables. Hence, α_1 and α_2 were arbitrarily fixed (α_1=α_2=.75) without a great loss of generality.

(2) Calculate an average deduct value for each distress type. This was done directly from Tables 48 and 49. For example, for rutting in flexible pavements,

TABLE 54 FLEXIBLE PAVEMENT UTILITIES FOR VARIOUS FREQUENCIES OF RURAL (α_1) AND SEGMENT (α_2) REHABILITATION

α_1	α_2	Cracking	PRS	Rutting	Failures/ Mile	Alligator Cracking	Flushing	Ravelling	Corrugating
.80	.80	.77	.59	.52	.20	.19	.17	.16	.12
.80	.75	.72	.56	.50	.25	.24	.21	.20	.15
.80	.67	.63	.49	.44	.33	.33	.28	.27	.20
.75	.80	.76	.58	.54	.20	.19	.17	.16	.12
.75	.75	.71	.55	.51	.24	.24	.20	.19	.15
.75	.67	.64	.49	.45	.32	.31	.27	.26	.20
.67	.80	.75	.56	.56	.19	.19	.16	.15	.12
.67	.75	.70	.53	.53	.24	.23	.21	.20	.15
.67	.67	.62	.46	.46	.32	.31	.28	.26	.20

1 mile = 1.6 km.

TABLE 55 RIGID PAVEMENT UTILITIES FOR VARIOUS FREQUENCIES OF RURAL (α_1) AND SEGMENT (α_2) REHABILITATION

α_1	α_2	Spalling	Pumping	Surface Deterio-ration	Cracking	PRS	Failures/ Mile	Faulting
.80	.80	.85	.84	.81	.78	.55	.20	.18
.80	.75	.86	.85	.82	.74	.51	.25	.23
.80	.67	.85	.85	.82	.65	.45	.33	.30
.75	.80	.86	.85	.82	.78	.54	.20	.19
.75	.75	.85	.85	.82	.73	.50	.24	.22
.75	.67	.85*	.87*	.81	.65	.45	.32	.30
.67	.80	.85	.85	.82	.77	.53	.19	.18
.67	.75	.87	.85	.83	.73	.50	.24	.22
.67	.67	.85	.84	.82	.64	.44	.32	.30

*indicates variables which are interchanged in order of importance.

$$\text{Rut} = (0+2+5+5+7+10+10+12+15) \div 9 = 7$$

If this value differs significantly from the median (in this case also 7), some reconciliation may be attempted. This problem occurs only for spalling in rigid pavements, and the solution chosen here is to pick an "average deduct value" intermediate to the mean and median. The average deduct values are present in Tables 56 and 57.

(3) Determine the relative severity ranking. This was done by multiplying the deduct values from step (2) by the appropriate utilities from step (1). Note that some distresses fit more than one category; for example, both "cracking" and "alligator cracking" are listed in Table 54. In general, the more specific category was allowed to stand on its own, but was also included in the more general category. Hence, in the example, alligator cracking is counted twice, once by itself and once along with all other types of cracking. Another way of saying this is that if all of the cracking in a pavement is alligator cracking, the effective utility is $U_{\text{cracking}} + U_{\text{alligator cracking}}$, whereas if all the cracking is transverse, the effective utility is U_{cracking}. This has the effect of making alligator cracking more important than other types of cracking in flexible pavements.

Note also that the utility for Pavement Rating Score (PRS) is included in all distress types, because it is a summary distress score.

The resulting severity ranking, from most to least severe, and using $\alpha_1=\alpha_2=0.75$, is:

Flexible Pavement Distress:

(1) Failures/mile.

(2) Alligator cracking.

(3) All other cracking.

(4) Flushing, Ravelling, Corrugation (very nearly equal).

(5) Rutting.

(6) Patching.

Rigid Pavement Distress:

(1) Pumping.

(2) Surface Deterioration.

(3) Spalling.

TABLE 56 AVERAGE DEDUCT VALUES FOR FLEXIBLE PAVEMENT DISTRESS

Distress	Deduct Value	Distress	Deduct Value
Failures/mile	30	Longitudinal Cracking	11
Alligator Cracking	15	Transverse Cracking	10
Raveling	13	Patching	8
Flushing	13	Rutting	7
Corrugations	13		

1 mile = 1.6 km.

TABLE 57 AVERAGE DEDUCT VALUES FOR RIGID PAVEMENT DISTRESS

Distress	Deduct Value	Distress	Deduct Value
Pumping	40	Y-Cracking	19
Failures/mile	30	Crack Spacing	16
Surface Deterioration	24	Transverse Cracking	16
Spalling	19	Longitudinal Cracking	15
Faulting	19		

1 mile = 1.6 km.

(4) Failures/miles.

(5) Cracking.

(6) Faulting.

These rankings are similar to those of Reference 30 (Table 53), provided that the difference categorization of distress types is taken into account.

APPENDIX C

EVAULATION OF DISTRESS MODELS

FLEXIBLE PAVEMENT

The commonly recognized major distresses for flexible pavements can be identified as fatigue cracking, low-temperature cracking, rutting and reduced skid resistance. VESYS A can predict the first three distress types. Reduced skid resistance, however, needs to be analyzed separately.

VESYS A is an improved version of Program VESYS IIM, a distress model that has been discussed in detail (Refs. 27[1], 28[2]). The capabilities added to VESYS IIM to produce VESYS A were:

1. Seasonal modification of material properties.

2. Incremental breakdown of the axle load distribution by tire radius and corresponding tire pressure.

3. Addition of a low-temperature cracking model.

The details of these revisions and the improvements in the idealization of flexible pavements are discussed in Reference 29.

The term PDMAP stands for Probabilistic Distress Models for Asphalt Pavement. The distress models included are for fatigue cracking and rutting. The low temperature cracking distress model is actually a separate computer program called Program COLD, but it was an integral part of NCHRP Project 1-10B research and was presented as a part of the PDMAP system (Ref. 39)[3]. Both the fatigue cracking distress and the rutting models in PDMAP are based on multiple regressions on data from

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[1]Rauhut, J.B., J.C. O'Quinn, and W.R. Hudson, "Sensitivity Analysis of FHWA Structural Model VESYS II", Report No. FHWA-RD-76-24, March 1976.

[2]Kenis, W.J., "Predicted Design Procedures - A Design Method for Flexible Pavements Using the VESYS Structural Subsystem", Proceedings, Fourth International Conference on Structural Design of Asphalt Pavements, Volume I, August 1977.

[3]Finn, F.N., C. Saraf, R. Kulkarni, K. Nair, W. Smith, and A. Abdullah, "Development of Pavement Structural Subsystems", Final Report NCHRP Project 1-10B, February 1977.

the AASHO Road Test, but depend on an elastic layer structural model to predict needed pavement responses. Complete descriptions of Program PDMAP, Program COLD, and their development are included in Reference 39.

VESYS A is a sophisticated computer code that accepts some twenty-three control variables and forty-four independent variables describing a flexible pavement structure, the traffic loadings it endures, and (through input of pavement temperatures and seasonal materials characterization) the environment in which it exists. Using this input, it then predicts fatigue cracking, rut depth, slope variance, present serviceability index and expected life as functions of time correlated to truck traffic.

The following paragraphs describe how PDMAP and VESYS A predict the major distress types for flexible pavement (Refs. 27, 18). The stiffer base layer in the composite pavement underneath the asphalt concrete surface can sometimes force the neutral axis of the stress diagram to fall below the surface layers (when subjected to a vertical load), which means that the bottom fiber of the AC surface will be in compression. For such instances, the fatigue analysis on the asphalt concrete surface will no longer be valid; instead it should be conducted at the base, or the stiffer layer assuming that when a fatigue crack is generated at the bottom of the base layer, it will propagate to the surface as a reflection crack.

Rutting

A diagram of the VESYS IIM pavement analysis and design system is shown in Figure 60. As can be seen, the input variables arrayed vertically at the left side of the diagram are partially stochastic and partially deterministic. Material properties inputs are fit to a curve using a Dirichlet series formulation so that the material characteristics are expressed in equation form for ready mathematical manipulation. The middle portion of the diagram shows a stationary load subprogram and a repetitive load subprogram. The stationary load subprogram is basically elastic layer theory applied to calculate stresses, strains, and displacement at the top and bottom of the asphaltic concrete layer, at the top and bottom of the base layer, and at the top of the subgrade layer. When a unit load is applied to a specified loaded area at the surface, one solution is then obtained for each load duration using the set of stiffnesses obtained for the three layers from their creep-compliance vectors. Calculated displacements at the top of the surface are then expressed in a Dirichlet series form as a function of load duration or time. Radial strains at the bottom of the surface layer are similarly

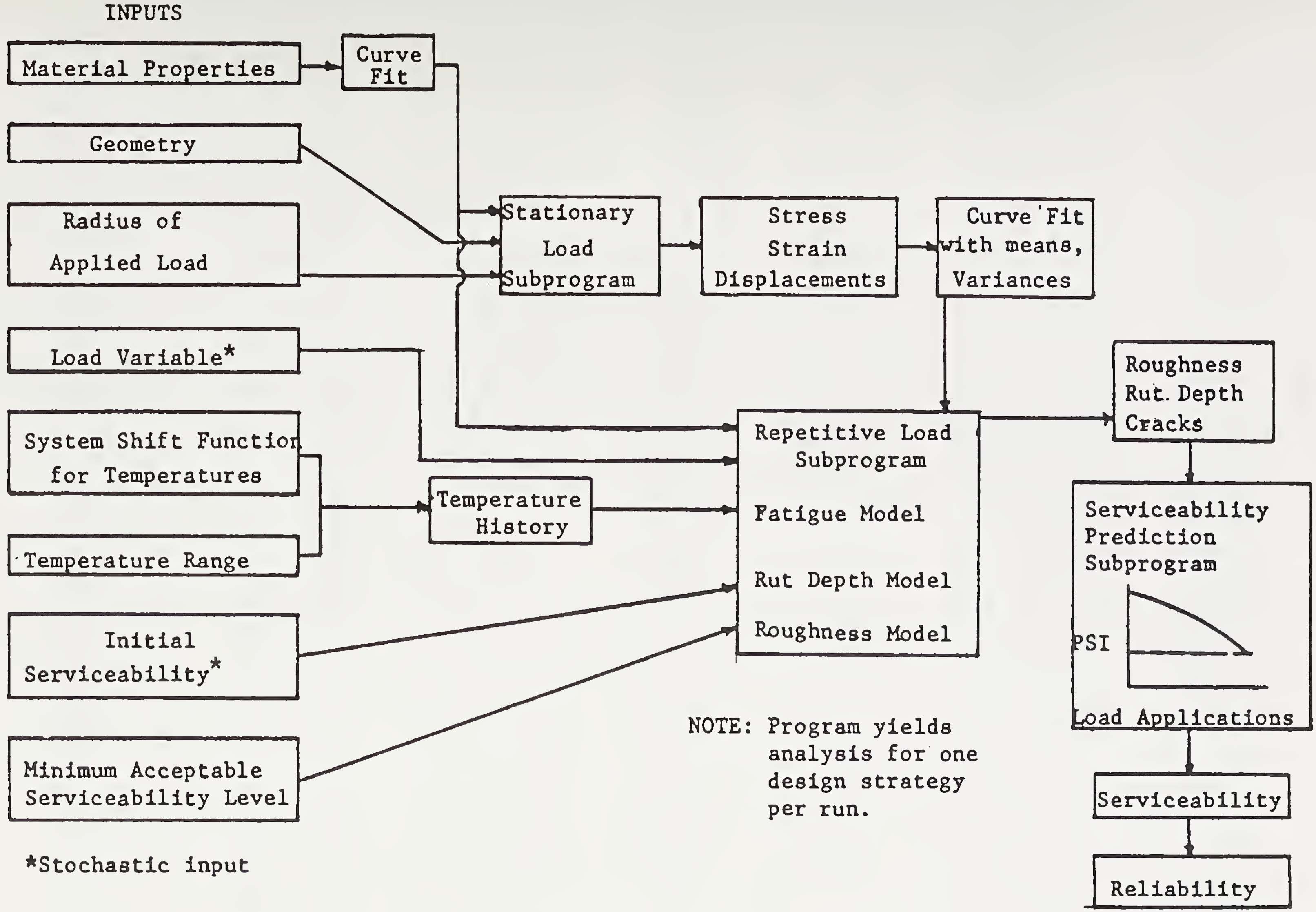

Figure 60 VESYS IIM pavement analysis and design system (Reference 30)

expressed as a function of time. The result at this stage is a set of equations representing total displacements at the surface and radial strains at the bottom of the surface layer, each as a function of loading time to reflect analytically the viscoelastic nature of the pavement materials.

A second series of similar static solutions are then run using values of layer elastic moduli derived by revising creep compliance relationships to omit permanent strains (basis is repetitive load testing to determine permanent strain as a function of number of loads applied). In explanation, the creep compliance values input to the computer program are based on total strains which are a combination of elastic and permanent strains.

The revised creep compliance values are based on elastic strains alone. We now have a second set of equations describing elastic or recoverable displacements. The differences of displacement at the surface between the two series of calculations may be taken to be the permanent strain or rutting at different load durations for one load cycle.

As permanent strain changes with number of load cycles applied, it is reasonable to calculate permanent deformation for various numbers of load applications so that displacements at the surface may be expressed as a function of both load duration and number of cycles.

Having the actual load duration and a time-temperature shift relation for the asphaltic concrete surface layer, a revised load duration that considers actual load duration and temperature effects on the creep compliance can be obtained. Application of these revised load durations to the sets of equations previously discussed also imposes the same revised load durations on the creep-compliance vectors for the base and subgrade, but the effect is minor as creep is small in these materials.

Sufficient information has been accumulated in the storage arrays at this point in the program from which to calculate the permanent deformations that occur at the surface and under the center of load. Separate calculations are made for each interval of time (usually one month) with its separate value of temperature specified and the corresponding number of loads or traffic during that time interval.

The analysis moves forward in time, using consistent revised load duration values for the temperature data, the number of cycles that have already occurred at the beginning of the time interval and those that will occur during the time interval to predict the accumulation of permanant displacements (rutting) at the surface since construction. Printout of these values may be made at points in time specified by the

computer program input TRANDØM.

The prediction of slope variance required for the AASHO equation for present serviceability index is obtained by statistical means on the basis of the stochastic input variables according to the following equation:

$$E[SV] = \frac{2B}{C^2} \cdot V[GNORMAL]^2 \left(E[RD]^2 + Var[RD]\right) \qquad (21)$$

where:

E[SV] ≈ Expected slope variance (radians x 10^6)

= mean slope variance

B = Materials variable called CØRLCØEF for which the value 1 may be used

C = Material variable called CØRLEXP

V[GNORMAL] = Coefficient of variation for the array of curve-fitted Dirichlet coefficients for normal time-dependent deflections at the surface as a function of load duration.

E[RD] = Mean rut depth

Var[RD] = Variance of rut depth

The variance of slope variance is predicted as:

$$Var[SV] \simeq \frac{16B^2}{C^4} \cdot V[GN\text{Ø}RMAL]^4 \cdot E[RD]^2 \cdot Var[RD] \qquad (22)$$

The very complex and laborious derivations of these equations are based on probability theory and appear in Reference 31.[1]

The rutting model for PDMAP predicts seasonal rate of rutting for permanent deformation per equivalent load application as a function of the following variables:

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[1]Soussou, J.E., F. Moavenzadeh, and H.K. Findakly, "Synthesis for Rational Design of Flexible Pavements, Part II", FHWA Contract No. FH-11-776, January 1973.

1. The elastic deflection at the surface of the pavement under an 18-kip (80 kN) axle load.

2. The vertical compressive stress of the asphalt concrete under an 18-kip (80 kN) axle load.

3. The total equivalent number of 18-kip (80 kN) single axle loads up to and including the season for which rate of rutting is to be calculated.

The model considers seasonal changes in the elastic constants used for the various layers and takes these into consideration as it accumulates rutting with time. The rutting predictions are displayed at different confidence levels on the basis of the stochastic characteristics of the elastic materials characterizations.

Fatigue

Fatigue calculations for estimating damage due to cracking are considerably less complex than those for rutting. Damage may be accumulated according to Miner's Hypothesis as follows:

$$D_j = \sum_{i=1}^{j} \frac{n_i}{N_i} = \sum_{i=1}^{j} \frac{\varepsilon_i^{K_{2_i}}}{K_{1_i}} \lambda_i (t_i - t_{i-1}) \qquad (23)$$

where:

n_i = Number of load cycles during the ith time interval

$= \lambda_i (t_i - t_{i-1})$

N_i = Number of cycles to failure under temperature and strain conditions of the ith time interval

$= K_{1_i} \left(\frac{1}{\varepsilon_i}\right)^{K_{2_i}}$

D_j = Mean damage index at time t_j

ε_i = Mean strain during the ith time interval

λ_i = Traffic rate during the ith time interval

t_i = The ith element of the TRANDØM array, $t_o = 0$

K_{1_i} = Fatigue relation coefficient for ith time interval in terms of temperature

K_{2_i} = Fatigue relation exponent for ith time interval in terms of temperature

The fraction of damage or cracking induced during each time interval is accumulated. Cracking failure is considered to occur when $D_j = 1$.

The coefficient K_1 and the exponent K_2 that define fatigue life in relation to initial strain at the bottom of the surface layer are temperature dependent and different values are input for each temperature value used.

As the fatigue materials characteristics are treated stochastically, the actual formulation used in VESYS IIM is:

$$E[D_j] = \sum_{i=1}^{j} n_i \; E[\frac{1}{N_i}] = \sum_{i=1}^{j} \lambda_i \, (t_i - t_{i-1}) \; \frac{\varepsilon_i^{K_{2_i}}}{K_{1_i}} + \frac{\text{Var } K_{1_i}}{(K_{1_i})^2} - \frac{\text{Cov }[K_{1_i}, K_{2_i}] \cdot {}^1{}_n \, \varepsilon_i}{K_{1_i}} + \frac{\text{Var } [K_{2_i}] \; ({}^1{}_n \, \varepsilon_i)^2}{2} \tag{24}$$

$$+ \frac{K_{2_i} \, (K_{2_i} - 1) \; \varepsilon_i^{(K_{2_i} - 2)} \text{Var}[\varepsilon_i]}{2K_{1_i}}$$

Where $\text{Cov}[K_{1_i}, K_{2_i}]$ = covariance of K_{1_i} and K_{2_i}.

This is the expected value of the Taylor series expansion for D_j about the means of K_{1_i}, K_{2_i} and ε_i. In this expression, the ε_i are assumed to be independent of the K_{1_i} and the K_{2_i}. The variance of D_j ($\text{Var }[D_j]$) can be obtained in a similar fashion, taking the second moment about the mean of the Taylor series expansion.

It is assumed that D_j is the value of a normally distributed random variable with mean $E[D_j]$ and variance $\text{Var}[D_j]$ as indicated in Figure 61. The cumulative distribution function (c.d.f.) for this variable at a point D_o is:

$$F(D_o) \int_{-\infty}^{D_o} f(t)\,dt = (2\pi \text{Var}[D_j])^{-1/2} \int_{-\infty}^{D_o} e^{-} \frac{(t-E[D_j])^2}{2 \text{ Var } [D_j]} \, dt \tag{25}$$

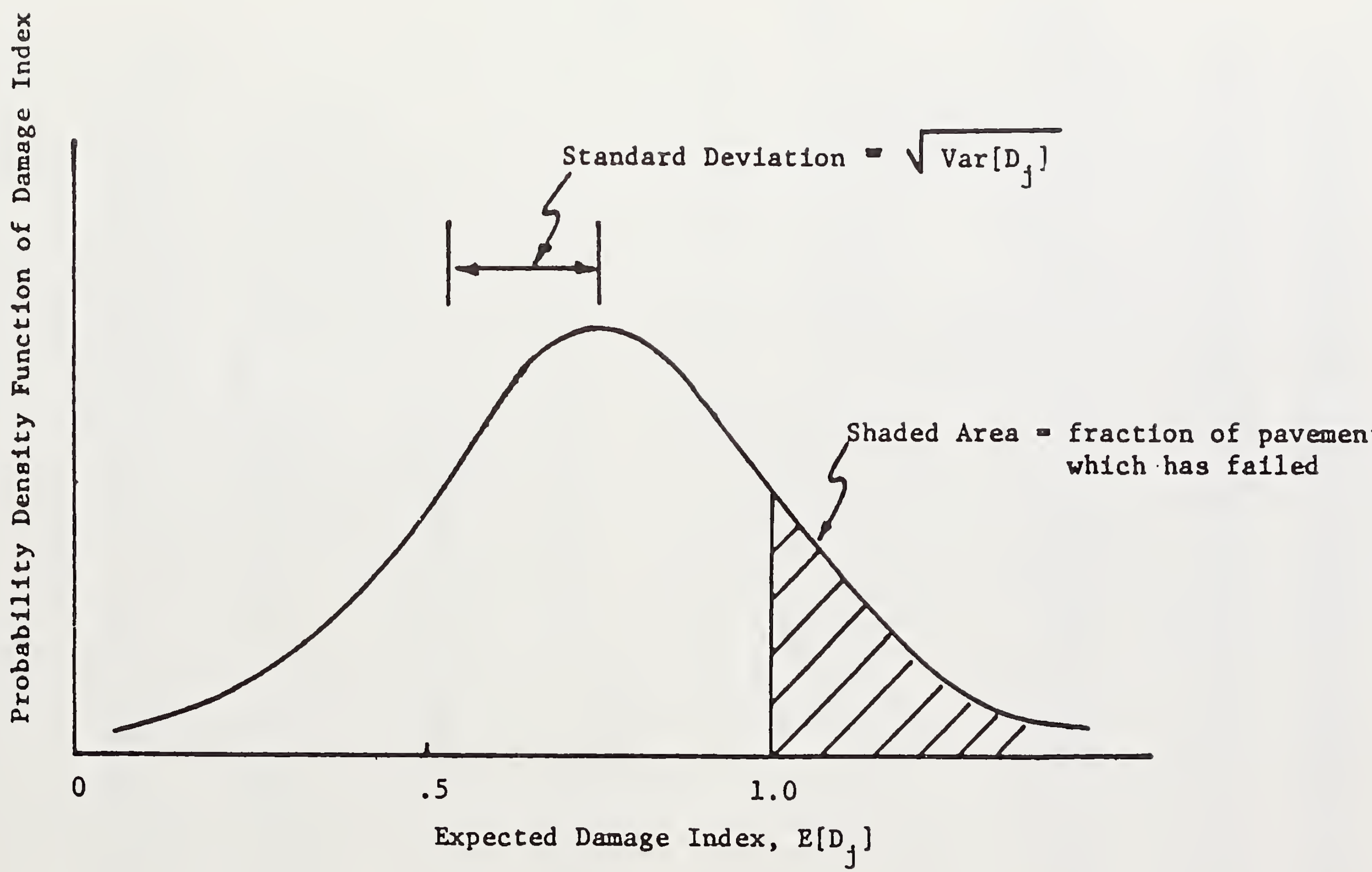

Figure 61 Normal predictability density function used to predict area cracked (Reference 30)

where:

$f(t)$ is the probability density function (p.d.f.) of damage at the point t (assumed normal), and

$F(D_o)$ is the c.d.f. of damage at the point D_o

$F(D_o)$ can be interpreted as the probability that the damage index of an arbitrary square yard of the pavement surface is less than or equal to D_o. Hence, under Miner's Hypothesis, F(1) is the probability that the specimen has not failed (i.e. does not exhibit cracking distress). Therefore:

$$C = 1000[1-F(1)] \tag{26}$$

where:

C is the expected area cracked in square yards per 1000 square yards.

The present serviceability index (PSI) is calculated from a stochastic version of the classical AASHO equation that yields the expected value of PSI as follows:

$$E[PSI] = 5.03 - 1.91\ E[\log_{10}(1 + SV)] - .01E[\ C + P] - 1.38\ E[RD^2] \tag{27}$$

$$E[PSI] \simeq 5.03 - 1.91\ \log_{10}(1 + E[SV]) - \frac{Var[SV].\ \log_{10}e}{2\ (1 + E[SV])^2}$$

$$- 01(E[C])^{1/2} - 1.38\ (E[RD]^2 + Var[RD]) \tag{28}$$

where:

SV = slope variance, radians x 10^6

RD = rut depth, inches

C = area of cracking, sq yds/1000 sq yds

The area of patching and the variation of cracking are assumed to be zero in the program VESYS A.

The fatigue distress model for PDMAP is very similar to that used in most fatigue predictions, except that an additional term has been added to take into account the effects of asphalt concrete stiffness. The resulting equation is as follows:

$$N = K_1 \left(\frac{1}{\varepsilon}\right)^{K_2} \cdot \left(\frac{1}{|E^*|}\right)^{K_3} \qquad (29)$$

where:

N = Number of Equivalent 18-kip single axle loads to first crack,

K_1, K_2, and K_3 are fitting coefficients,

ε = Maximum tensile strain in lower fibers of asphalt concrete, and

$|E^*|$ = Complex modulus of asphalt concrete.

The approach taken to arrive at the fitting coefficients for Equation (29) is aimed at overcoming the known limitations of laboratory beam testing and for correlating the results of such testing with field experience. The approach taken was to conduct a multiple regression analysis on laboratory beam test data developed by Monismith et al (Ref. 40)[1] and other test data developed during the NCHRP 1-10B project. The resulting coefficients for K_1, K_2, and K_3 were 14.82, 3.291, and 0.854, respectively. The resulting equation grossly underpredicts fatigue life (as does almost all data from laboratory fatigue tests data). "Shift factors" were developed on the basis of comparisons to AASHO Road Test results to extrapolate those data to field conditions. The results of this study indicated that it took 13 to 18 times the number of strain replications predicted by the laboratory fatigue tests to actually propagate cracks to the surface of a real pavement. These results are fairly consistent with those resulting from wheel tracking tests conducted by Shell Laboratories (Ref. 41).[2]

As a relationship between dynamic modulus and temperature was known for the asphalt concrete used in the AASHO Road Test (Ref. 42), it was possible to compare the PDMAP fatigue cracking relationships on the basis of varying stiffness to those developed by Rauhut, et al (Ref. 27), on the basis of temperature. It was found that the variation in predicted fatigue life with stiffness or temperature was much smaller than the variations found by other researchers for seven different

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[1]Monismith, C.L., J.A. Epps, D.A. Kasiancheck and D.B. McLean, "Asphalt Mixture Behavior in Repeated Flexure", Institute of Transportation and Traffic Engineering Report No. TE-70-5, University of California, Berkeley, 1972.

[2]Van Dijk, W., "Practical Fatigue Characterization of Bituminous Mixes", paper presented at the Annual Meeting of the Association of Asphalt Paving Technologists, Phoenix, Arizona, February 1975.

materials. Although the variation in predicted fatigue life with material stiffness seems to be much more limited than that found by other researchers, the fatigue relationship at 70°F (21.1°C) is quite close to that one selected by Rauhut, et al (Ref. 27), as a mean fatigue damage relationship for the sensitivity analysis on VESYS IIM.

The structural model itself has several useful features as listed below:

1. The stiffnesses for all layers are input as variables. The stiffness for the asphalt concrete is a function of temperature, while the stiffnesses of base, subbase, and subgrade layers are functions of either lateral stress or sum of principal stresses at the selection of the user. This allows iterative solutions to compatibility between stiffness and stress in the soil layers.

2. The temperatures of the asphalt layers are calculated from environmental data based on the Barber method (Ref. 43).

3. Coefficients of variation for the stiffnesses for the various layers are furnished and used to arrive at the probabilistic response predictions. PDMAP like VESYS A produces families of predicted responses for each "season" of the year and uses these families of solutions in a systematic manner to predict distresses with time.

It is believed that the rutting distress model has some limitations that restrict its value to this research, such as:

1. The regression model is based entirely on elastic material properties and elastic responses and includes no permanent deformation characterization of the materials at all. The consideration of the permanent deformation characteristics of materials in this regression model are entirely implicit and would only apply directly to those materials in the pavements of the AASHO Road Test.

2. The three regression coefficients in the rate of rutting in the prediction model are based entirely on AASHO Road Test materials and conditions and, according to the authors of Reference 39, require reestablishment for use under other conditions.

The fatigue cracking distress model as used in the program also requires establishment of three regression coefficients. However, similar fatigue damage relationships have to be established regardless of the model considered if realistic results are to be expected. The basic nature of each of the four fatigue cracking distress models under consideration are essentially the same.

Low Temperature Cracking

Program COLD is a sophisticated procedure which estimates temperature and thermal stresses in the pavement, and predicts when low-temperature cracking is likely to occur. Temperatures in the pavement are estimated at two-hour increments for each day included in the study. Strength-temperature relationships are provided as input to the program and are compared with the thermally induced stresses at two-hour intervals. The program indicates the expected time at which low-temperature cracking will occur. The materials inputs for Program COLD include absorbitivity, emissivity, and convection coefficient of the surface; thermal conductivity of the material (both unfrozen and frozen); heat capacity of the material (unfrozen and frozen); dry density and moisture content of the materials; thicknesses of the layers; and both stiffness modulus and strength values as functions of temperatures.

The low-temperature cracking model used in VESYS A was developed by Haas and Hajek (Ref. 32)[1]. While this model does a relatively acceptable job of predicting low temperature cracking, it is based on multiple regressions for pavements in Canada alone and uses the five following parameters:

1. Age of pavement.
2. Thickness of the bituminous layers.
3. Subgrade soil type in a numerical code.
4. Winter design temperature.
5. Stiffness of original asphalt cement.

In summary, VESYS A is perhaps the most complete flexible pavement distress model in existence today and considers a broad range of material properties in its distress subsystems. Some of the input variables, such as those for the permanent deformation characterizations of materials, are relatively new to the engineering profession; however, these variables are not new to the project staff, since they developed the available data for these parameters in two previous contracts for the FHWA (Refs. 27 and 29).

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[1]Haas, R.C.G., "A Method for Designing Asphalt Pavement to Minimize Low-Temperature Shrinkage Cracking", Research Report 73-1, The Asphalt Institute, January 1973.

JOINTED CONCRETE PAVEMENT

The FHWA recently conducted a study involving an evaluation of performance requirements and capabilities of heavily trafficked conventional pavement types to provide premium pavements for zero-maintenance (Ref. 22). In this study, field surveys were conducted on 68 in-service, heavily trafficked pavements located in 13 states. According to survey results and from interviews with state highway and transportation personnel, the major distresses found that required or received maintenance for jointed concrete pavements were transverse cracking and faulting. Other distresses which have higher frequency occurrences and were rated moderate to severe include "D" cracking, joint filler extrusion and shoulder distress.

The model selected to analyze jointed concrete pavements is the JCP-1 computer program developed by Darter (Ref. 33)[1]. This model is based on two separate analyses. The serviceability/performance analysis will predict roughness which is a measure of the above listed distresses combined. The fatigue analysis predicts the occurrence of transverse cracks. The following paragraphs briefly summarize the JCP-1 design model.

Serviceability Performance Analysis

Data taken from twenty-five sections of the original AASHO Road Tests accumulated by the Illinois Department of Transportation over sixteen years, plus data from twelve other plain jointed concrete projects located around the United States were tested against the original AASHO equation as modified to include modulus of rupture, modulus of elasticity, and modulus of foundation support. Darter found that the standard error of the estimate (standard deviation of residuals) is .31 for log W, which is the logarithm of the number of 18-kip (80 kN) equivalent single axle loads, to reduce the initial serviceability index of a given value to a specific terminal serviceability index. This standard error is considerably larger than the error based on only the results from the limited two year road test, which was 0.22.

An approach similar to that used to develop the original AASHO Road Test Equation was used to develop a modified equation. Using data from the 25 sections of the original AASHO Road Test and data from 12 other plain jointed concrete pavements, the new serviceability index equation becomes:

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[1]Darter, M.I., "Design of Zero-Maintenance Plain Jointed Concrete Pavement, Vol. 1 - Development of Design Procedures", Report ZM-2-77, prepared by the Department of Civil Engineering, University of Illinois at Urbana-Champaign, prepared for the FHWA, June 8. 1977.

$$W'_{18} = [\rho \ln(\frac{3}{y} - 1) + \beta]10^{6} \tag{30}$$

where:

W'_{18} = total equivalent 18-kip (80 kN) single axle loads to reduce the serviceability index from P1 to P2.

$\beta = 50.08826 - 3.77485H + 30.64386 \quad H$

$\rho = 6.69703 + 0.13879H^{2}$

$$y = P2 + \frac{3.0}{(e)^{-\beta/\rho + 1}} - P1$$

P2 = terminal serviceability index

P1 = initial serviceability index

H = PCC slab thickness, inches

The above equation was extended to consider K - value and PCC modulus of rupture by using Westergaard's equation for edge loading and Spangler's corner equation. The final equation becomes:

$$\log_{10}W_{18} = \log_{10}W'_{18} + (3.893-0.706P2)\log \quad (\frac{F28}{690})\frac{4\ \log[\frac{8.789H^{0.75}}{M}]+0.359}{4\ \log[\frac{(Z)^{0.25}(0.540H^{0.75})}{M}]+0.359} \tag{31}$$

in which:

$M = 1.6a^{2} + H^{2} - 0.675H$

a = radius of applied edge load, inches

F28 = modulus of rupture used in design (28 day, 3rd point load adjusted for variability) = FF-C $(\frac{Fcv}{100})$FF

FF = means modulus of rupture at 28 days, 3rd point load, psi

Fcv = coefficient of variation of modulus of rupture, percent

C = 1.03, a constant representing a confidence level of 85 percent

Z = E/k

E = modulus of elasticity of PCC, psi

k = modulus of foundation support on top of subbase, psi

Predictions of load applications using the AASHO equation and Equation 30 are shown in Figures 62 and 63, respectively. Data obtained for Equation 30 show a better correlation than the original AASHO equation. Figure 64 shows the graphical solution of Equation 31.

Fatigue Damage Analysis

The development of the fatigue damage model is based on the following criteria:

1. Critical stress used for the fatigue analysis is located at the slab longitudinal edge midway between transverse joints.

2. Critical edge stresses caused by both traffic loads and thermal gradient curl are considered.

3. Both load and thermal curl stresses are computed using a finite element program.

4. The proportion of traffic occurring near the slab edge is used in the fatigue analysis.

5. Concrete strength changes with time and thus the fatigue analysis must be time dependent.

6. Fatigue "damage" is computed according to the Miner hypothesis.

The procedure used for the fatigue analysis is as follows:

(1) Select trial slab thickness

(2) Compute fatigue damage by the following equation:

$$D = \sum_{k=1}^{k=p} \sum_{j=1}^{j=2} \sum_{i=1}^{i=m} \frac{n_{ijk}}{N_{ijk}} \tag{32}$$

where:

D = fatigue damage

$\sum_{k=1}^{k=p}$ = accumulation of traffic from the 1st month to the final month

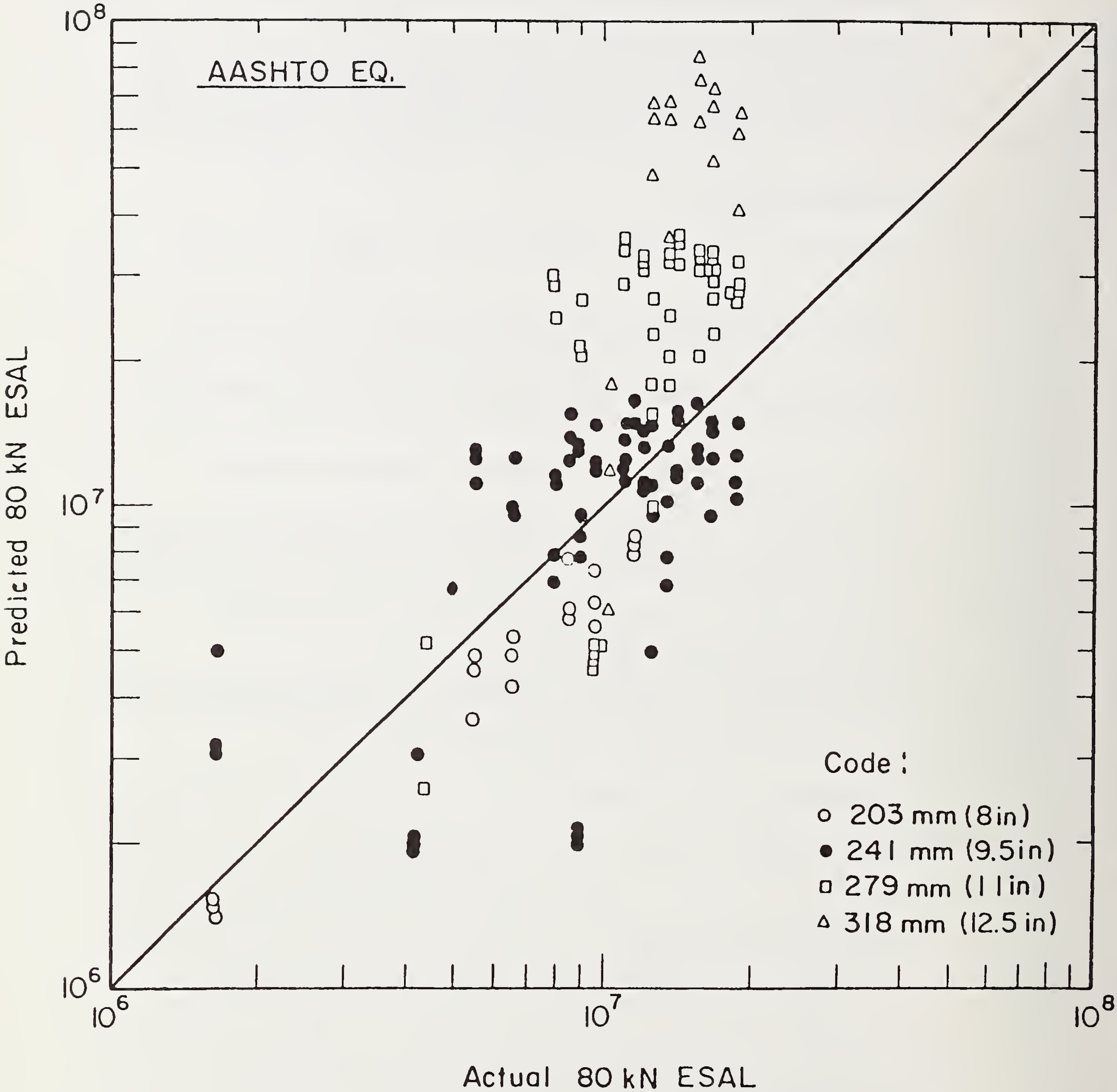


Figure 62 Predicted vs. Actual Equivalent 80 kN (18-kip) Single Axle Load Applications using AASHO Equation and 16 Years Data from AASHO Road Test Site (1958-1974) (Reference 8)

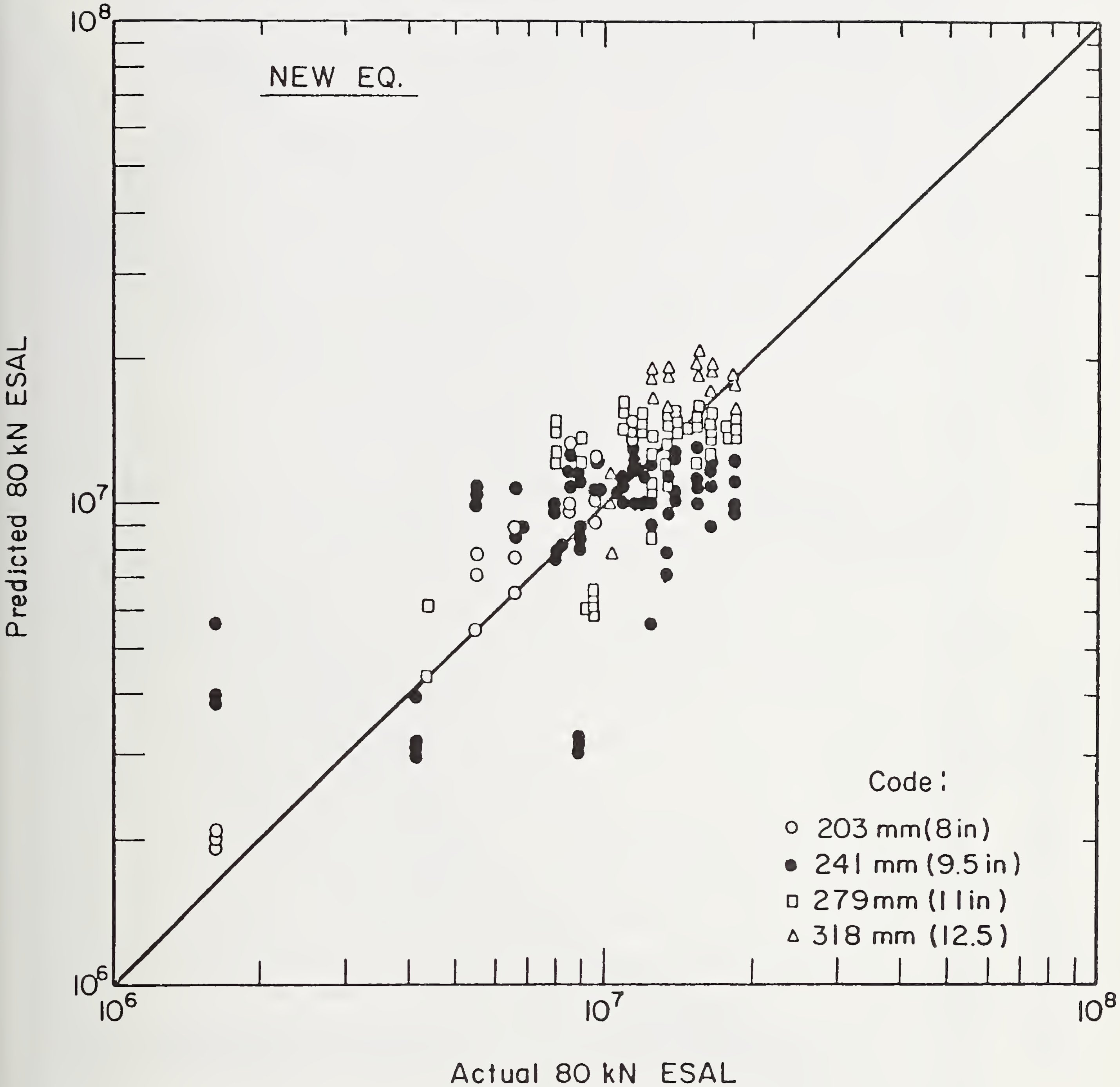
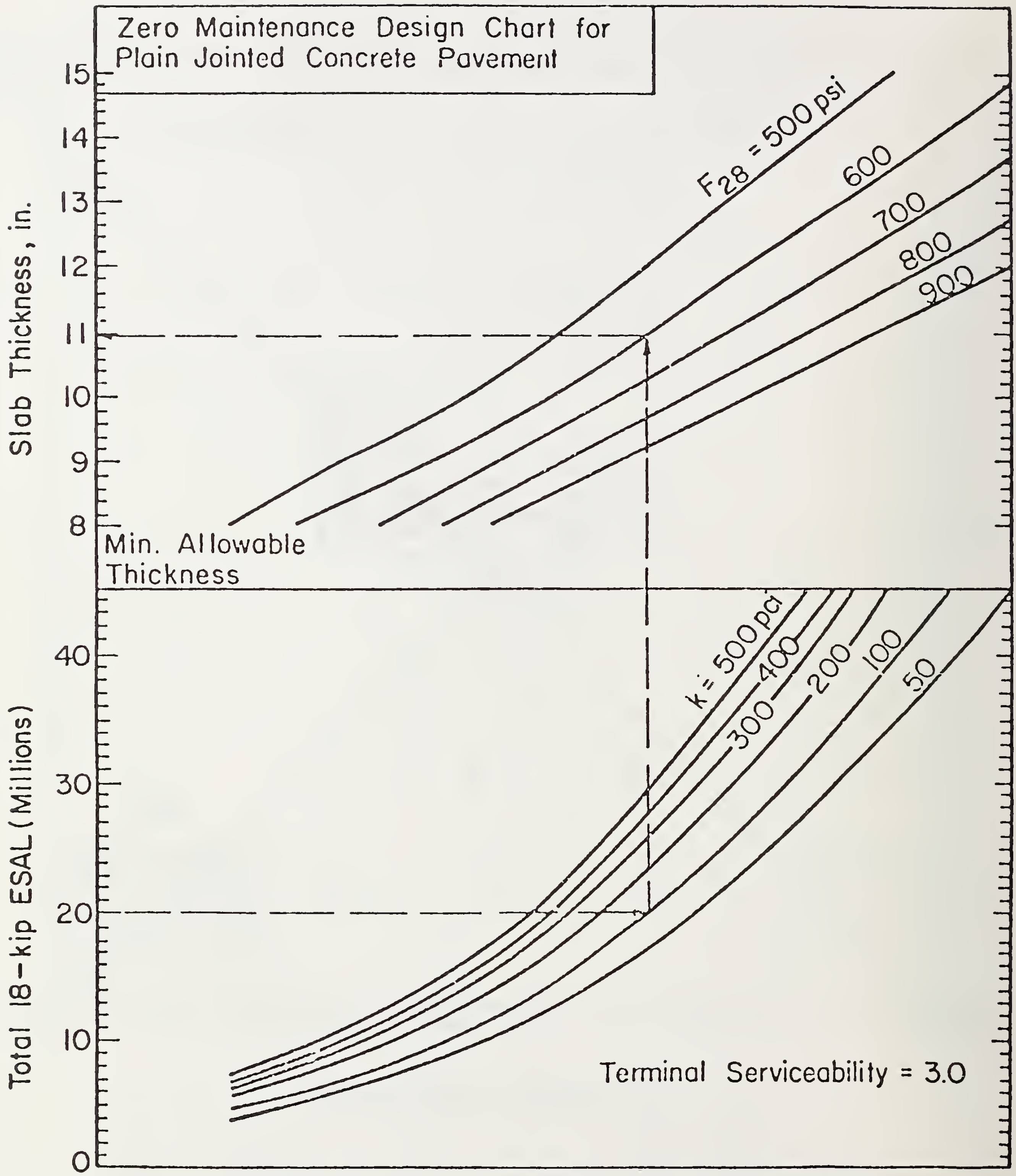


Figure 63 Accuracy of New Design Eq. 10
(Reference 8)



1 inch = 2.54 cm. 1 psi = 6.89 kPa 18 kip = 80 kN

Figure 64 Serviceability/Performance Zero-Maintenance Design Chart for Plain Jointed Concrete Pavement (Reference 8)

$\sum_{j=1}^{j=2}$ = accumulation of traffic for one day and night

$\sum_{i=1}^{i=m}$ = accumulation of total number of single and tandem axle load groups

n_{ijk} = actual load application

N_{ijk} = allowable load applications

(3) n_{ijk} is computed by the following equation:

$$n_{ijk} = (ADTm)(T/100)(DD/100)(LD/100)(A)(30)$$
$$(P/100)(C/100)(DM/100)(TF/100)(CON/100) \tag{33}$$

where:

ADTm = average daily traffic at the end of the specific month under consideration

T = percent trucks

DD = percent traffic in direction of design lane

LD = lane distribution factor, percent trucks in design lane in one direction

A = mean number of axles per truck

P = percent axles in ith load group

C = percent of total axles in the lane that are within 6 inches of the edge

DN = percent of trucks during day or night

TF = factor to either increase or decrease truck volume for the specific month

CON = 1 for the single axles, 2 for tandem axles

(4) N_{ijk} is computed using the following equation:

$$\log N_{ijk} = 16.61 - 17.61 \left(\frac{\sigma tot}{F}\right) \tag{34}$$

where:

F = modulus of rupture of PCC

$\sigma_{tot} = \sigma_{load} + R\ \sigma_{curling}$

R = adjustment factor for curling stress so that it can be combined with the load stress.

The curling stress, $\sigma_{curling}$, and the load induced stress, σ_{load} are calculated by regression equations in the computer program, which are based on analytical solutions from a finite element program.

The total accumulated fatigue "damage" from the opening of each project to traffic until the date of survey was computed using the computer program JCP-1. A plot of cracking index (ft/1000 ft^2) versus total accumulated fatigue "damage" is shown in Figure 65. The relationship between slab cracking and computed fatigue "damage" indicates that a correlation exists between transverse slab cracking and computed fatigue "damage" in the slab.

The amounts of probable transverse cracking for various accumulated "damage" determined from the curve fit through the data is as follows:

Fatigue Damage	(ft/1000 ft^2) Cracking Index	Example for L=15 ft Percent Slabs Cracked
10^{-5}	0	0
10^{-4}	0	0
10^{-2}	1.5	2
1	11	17
10	20	30
100	30	45

1 foot = 0.305 cm. 1 sq. ft. = .093 sq. m.

CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

The major distresses commonly found in continuously reinforced concrete pavement can be identified as fatigue cracking, low-temperature and shrinkage cracking, punchouts, spalling and steel rupture. Fatigue cracking due to repetitive vertical loads will be analyzed by the fatigue model

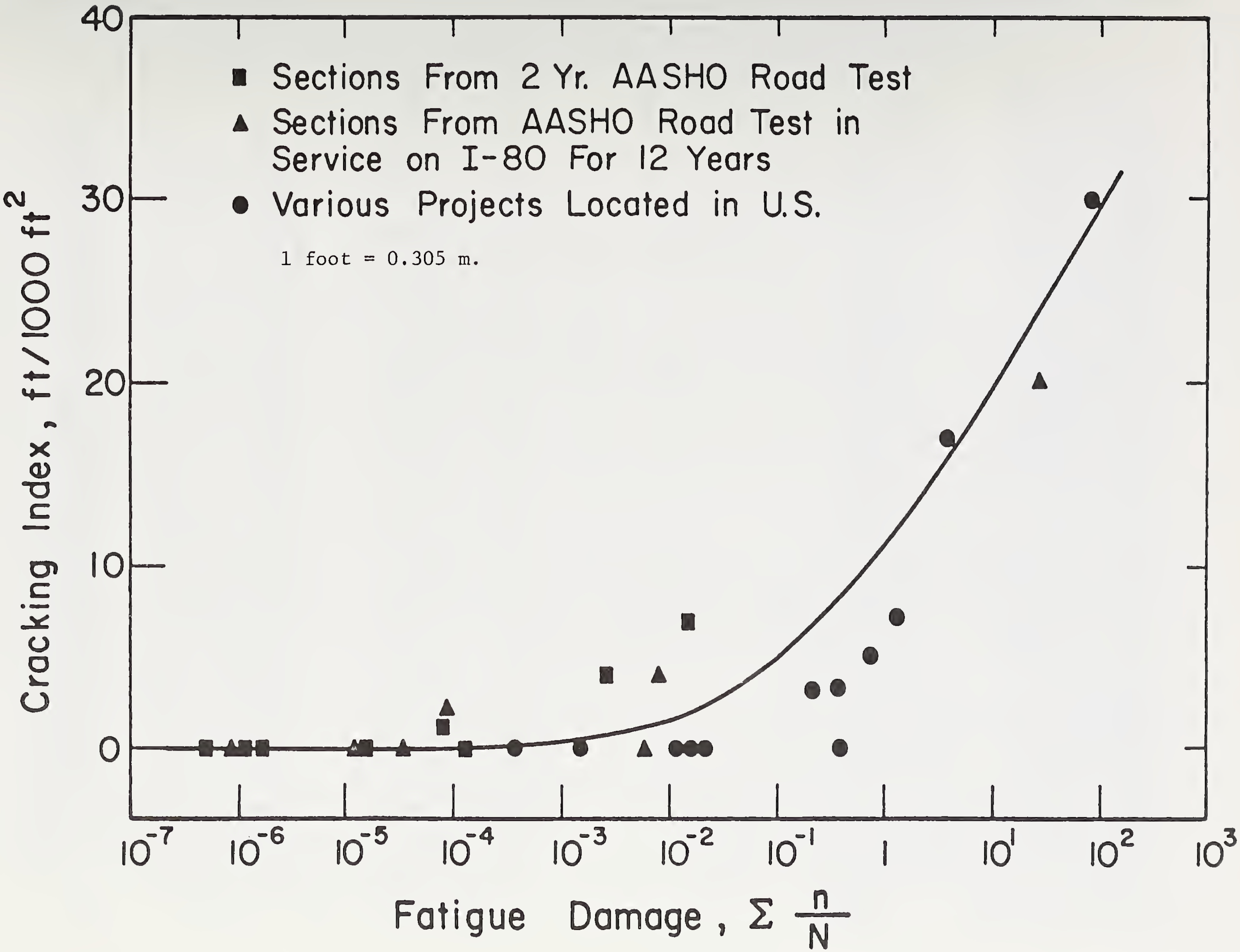


Figure 65 Effect of Fatigue "Damage" on Cracking of PCC Plain Jointed Concrete Pavements (Reference 8)

described in Reference 34[1] incorporated with the stress analysis by the layer program, ELSYM5 (Ref. 35)[2]. Steel rupture, low-temperature and shrinkage cracking will be predicted by the computer program CRCP-2 (Ref. 36)[3], an improved version of CRCP-1. Although there is no mechanistic model which specifically predict spalling and punchouts, results from CRCP-2 analysis which includes crack spacing and crack width can be correlated with these distresses. In general, wider cracks suffer crack spalling while close crack spacing induces punchouts.

Fatigue Analysis

The fatigue model in Reference 34 is based on the failure criteria of class 3 or class 4 cracking, as defined at the AASHO Road Test. The fatigue analysis consists of predicting axle applications to failure using the following equation:

$$N = 23{,}440 \ (f/\sigma)^{3.21} \qquad (35)$$

where:

N = number of axle loads until failure

f = flexural strength of concrete, psi

σ = computed stress due to design load, psi

The stresses needed for the fatigue analysis can be computed by elastic layered theory. An attribute of elastic layer theory is that all the layers in the pavement structure can be characterized individually and their separate effects on pavement responses studied. Stresses and strains predicted by elastic layer theory are spread with depth in a more realistic fashion than for the "plate models" and it is relatively

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[1]Treybig, H.J., B.F. McCullough, P. Smith and H. Von Quintus, "Overlay Design and Reflections Cracking Analysis for Rigid Pavements, Volume 1, Development of New Design Criteria", Final Report No. FHWA-RD-77-66, January 1978.

[2]Ahlborn, Gale, "Elastic Layered System with Normal Loads", Institute of Transportation and Traffic Engineering, University of California, Berkeley, May 1972.

[3]Ma, J.C.M., "CRCP-2, An Improved Computer Program for the Analysis of Continuously Reinforced Concrete Pavement", University of Texas at Austin, Master's Thesis, August 1977.

more economical to operate. The primary limitations of elastic layer theory are its inability to define any horizontal boundaries and to simulate directly the existence of variations in stiffness in the pavement structure, cracks in the surface, or voids under the surface layer. Despite these limitations, however, elastic layer theory offers very useful capabilities for prediction of bending stresses in pavement layers.

A recent study conducted by Schnitter (Ref. 37)[1] made a comparison of various layer programs. In his study, Schnitter operated computer codes for ELSYM5, LAYER5, LAYER15, LAYIT and BISAR for several typical problems over a range of conditions. All of the programs gave essentially the same results, usually within one percent and with a maximum difference of only three percent. The most economical program for use was LAYER5, but it is capable of handling only one wheel load at a time. Only ELSYM5 and BISAR can handle multiple loads. ELSYM5 is more economical to operate than BISAR and five layers are generally sufficient for the problems anticipated in this project. BISAR, however, can consider both variable friction at its interfaces and a horizontal load applied at the surface, so it may have utility for special studies. Based on the study of elastic layered programs ELSYM5 was judged to be the best overall elastic layer computer code for use on this project.

Low-Temperature and Shrinkage Cracking

The dimensional changes in a continuously reinforced concrete pavement caused by drying shrinkage of the concrete and temperature variation after curing will be investigated by the design method CRCP-2 (Ref. 36).

Figure 66 shows the geometric model used to develop the basic equations for the CRCP-2 design method. Due to the accumulated friction and the terminal treatments used in the construction, the slab model assumes an anchorage at each end so that the pavement within the anchorages will maintain a fixed length.

The difference in the thermal coefficients of the steel and the concrete together with the drying shrinkage of the concrete enable us to determine the internal stress in the reinforced slab. Figure 67 shows a typical thermal contraction model at a fully bonded zone on a frictionless base. Using the friction-movement characteristic of the slab and the

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[1]Schnitter, O., "Comparison of Stresses, Strains, and Deflections Calculated with Various Layer Programs", Pavement Design Course Term Project, University of Texas at Austin, Spring, 1977.

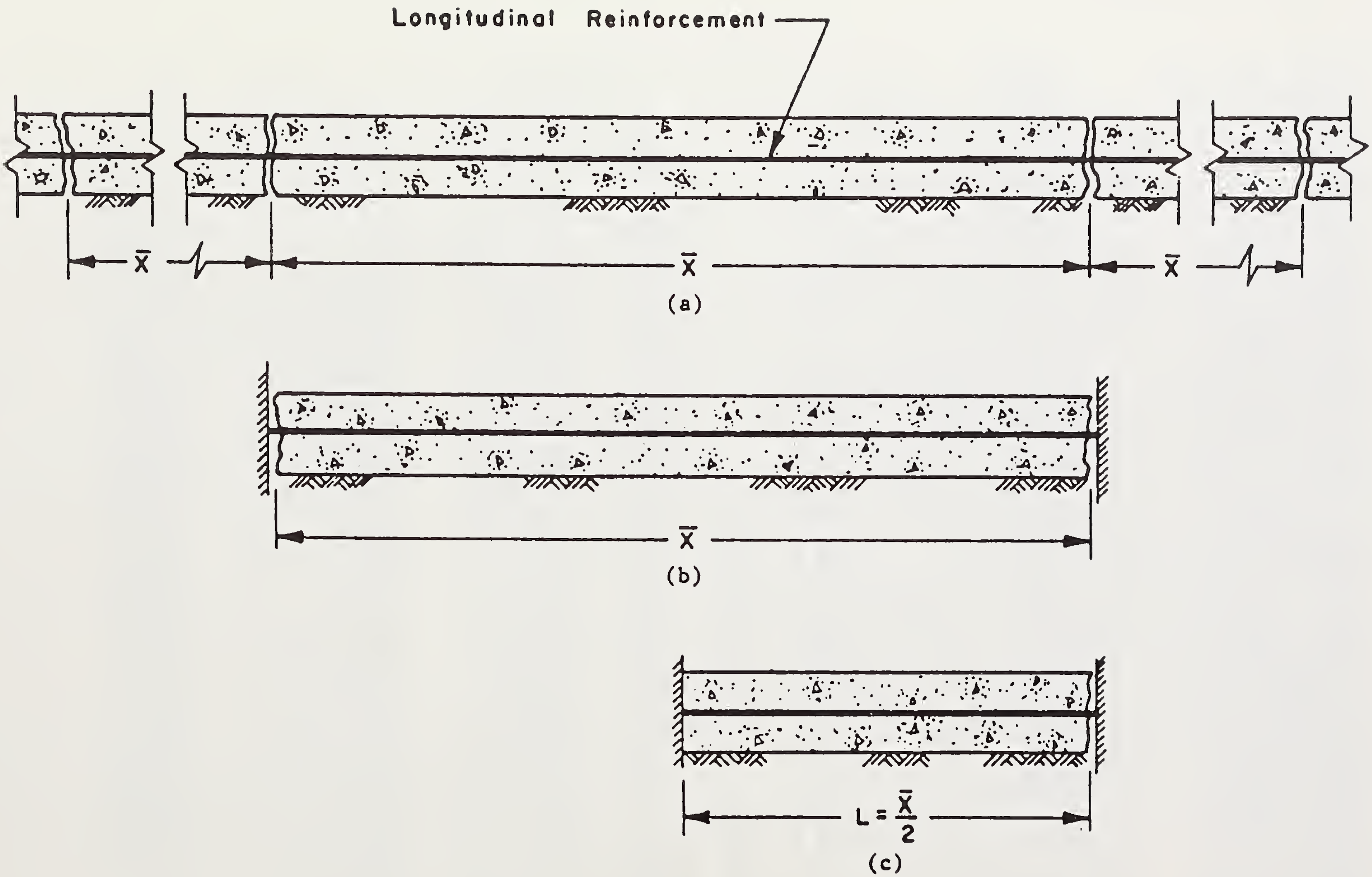
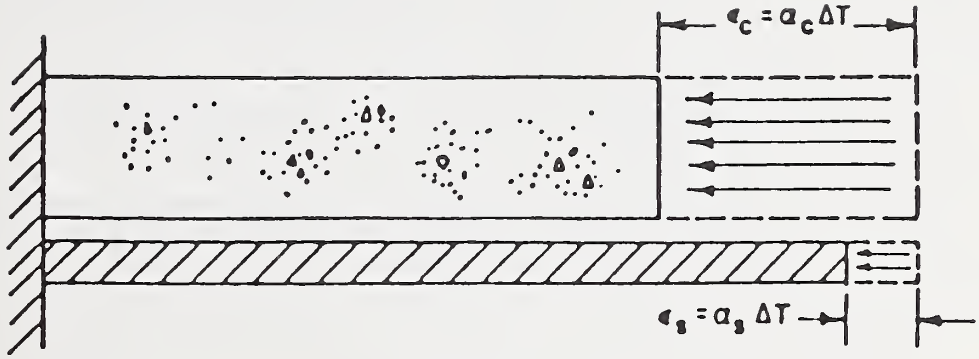
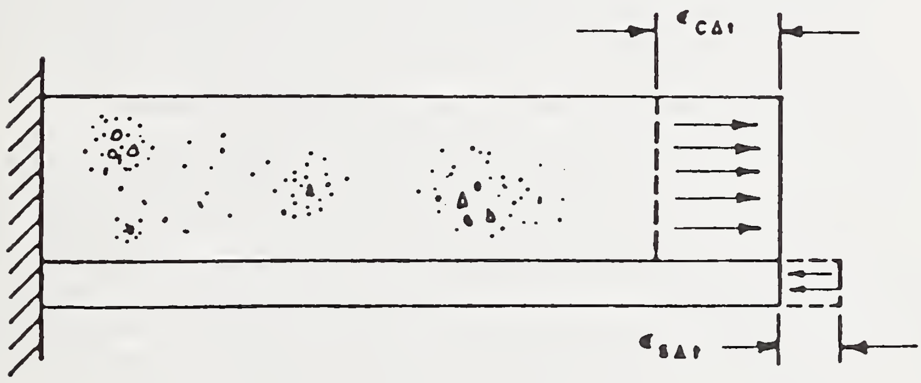


Figure 66 Continuously reinforced concrete pavement geometric model (Reference 36)



(a) Steel and concrete not bonded.



(b) Steel and concrete fully bonded.

Figure 67 Behavior of a reinforced slab subjected to temperature drop (Reference 36)

soil determined in laboratory experiments, the degree of restraint due to the soil frictional resistance can be estimated. By assuming equilibrium in the system, the stress of one material can be correlated to the stress of the adjacent materials. Finally, an incremental approach was adopted to predict the formation of the transverse cracks as a function of time by comparing the concrete stress with the strength of the concrete.

In the development of the model, the following assumptions were made:

(1) A crack occurs when the concrete stress exceeds the concrete strength, and after cracking, the concrete stress at the location of the crack is zero.

(2) The concrete and steel properties are linearly elastic.

(3) In the fully bonded sections of the concrete slab, there is no relative movement between the steel and the concrete.

(4) The force displacement curve which characterizes the frictional resistance between the concrete slab and the underlying base is elastic.

(5) Temperature variations and shrinkage due to drying are uniformly distributed throughout the slab, and hence, a one-dimensional axial structural model is adopted for the analysis of the problem.

(6) Material properties are independent of space.

(7) The effects of creep on concrete and slab warping are neglected.

Spacing of transverse cracks that occur in continuously reinforced concrete pavements is perhaps the most important variable affecting the behavior of the pavement. Relatively large distances between cracks results in a higher accumulation of drag forces from the subgrade due to frictional resistance, thus producing high steel stress at the crack and wide crack widths. Closer crack spacing reduces the frictional restraint, thus the steel stress and the crack width.

JOINTED REINFORCED CONCRETE PAVEMENT

The major distress found in jointed reinforced concrete pavements are similar to those found in CRC pavements. Punchouts however are more common in CRCP than in JRCP due to the close spacing between transverse cracks. Low temperature and shrinkage cracking will be analyzed with

the computer model JRCP-2 (Ref. 38)[1]. Fatigue cracking can be investigated concurrently with CRC pavement using the Layer Theory and the fatigue model discussed previously.

Low Temperature and Shrinkage Cracking

The mechanics of the dimensional change due to temperature and moisture variations in JRC pavement are similar to that in CRCP. The boundary conditions, however, are different. Program JRCP-2 developed by Rivero-Vallejo and McCullough (Ref. 38) is an updated version of JRCP-1 and uses many of the concepts developed for CRCP-2. Two models are necessary for the derivation of the basic equations to represent the behavior of the slab. Before the occurrence of the first crack, Model-1 assumes both ends free as shown in Figure 68. After the first crack has developed, Model-1 will change into Model-1 plus Model-2. The portion of the slab going from the crack to the free end will have one end restrained by the steel at the crack and the other end free. A problem of bond development length is present at the crack, because the steel requires some finite length to transmit the stress to the concrete. Both ends will contract, but the one with the steel, in a fixed condition, will have more restraint. Consequently, the point of zero movement will be more towards the crack side as represented in Figure 68. When the second crack develops, the portion of the slab from the crack to the mid-slab will be the same as the CRCP model as shown in Figure 68. JRCP-2 considers the frictional resistance of the subbase; the stiffness, tensile strength, and the shrinkage coefficient of the concrete; the temperature drops anticipated in time; the slab length; the percent reinforcement; the reinforcing bar diameter; the yield stress of the steel; the elastic modulus of the steel, the unit weight of the concrete; and the ages at which cracking is to be considered. Given this information, JRCP-2 theoretically will proceed with analysis until the first crack forms and then will restructure the problem for subsequent consideration for the formation of a second crack between the joint and the first crack. The output includes the time when the first crack is formed, concrete stress, steel stress, joint width, and crack width as a function of time, and the same data for second cracks if they are formed. A summary follows of the basic equations developed for each model.

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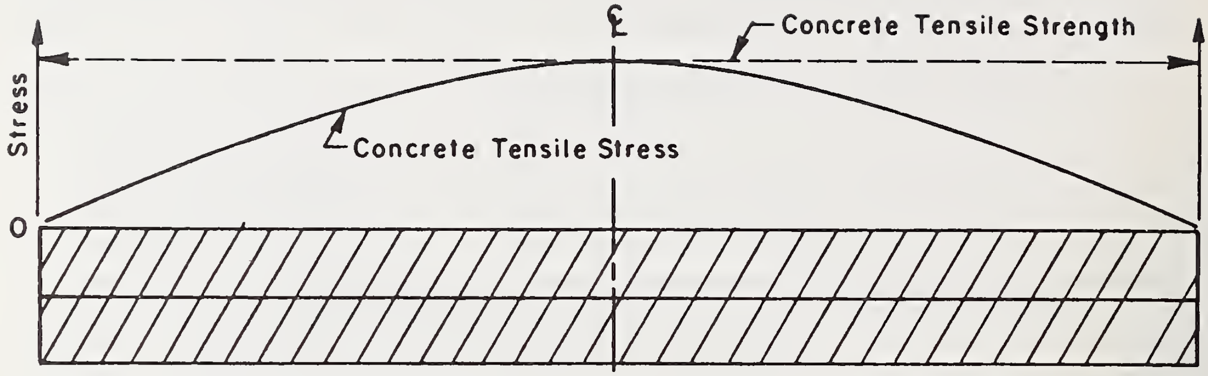
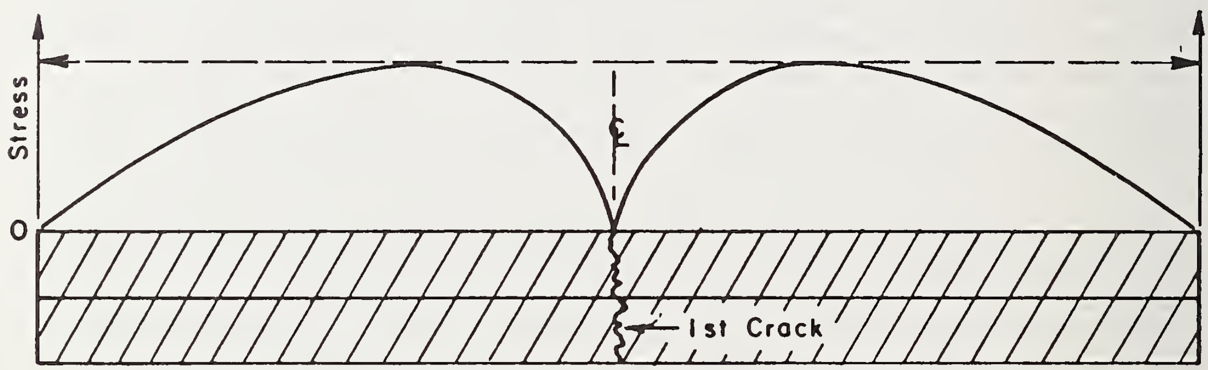
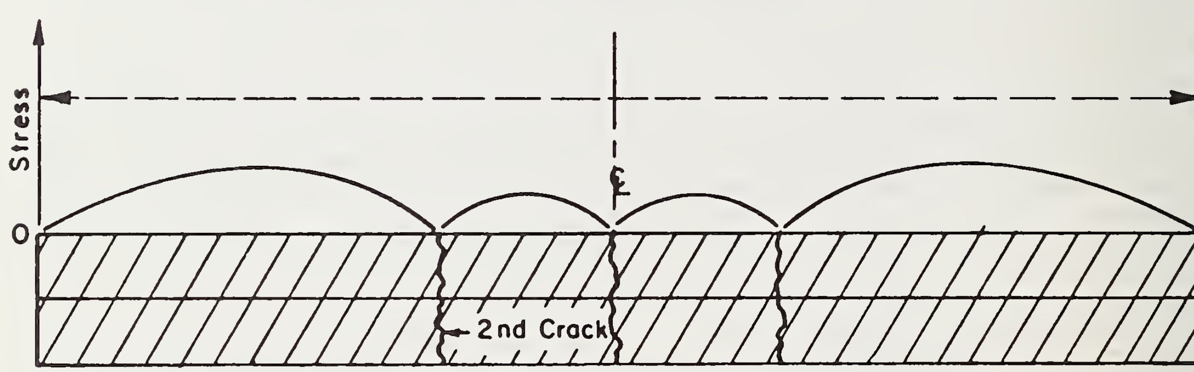
[1]Rivero-Vallejo and B.F. McCullough, "Drying Shrinkage and Temperature Drop Stresses in Jointed Reinforced Concrete Pavement", Report 117-1, Center for Highway Research, The University of Texas at Austin, August 1975.



(a) Concrete slab prior to cracking



(b) Concrete slab after first crack



(c) Concrete slab after second cracks

Figure 68 Crack occurrence in a JRC slab (Reference 38)

Model-1:

(1) Equilibrium

$$\sigma_c + \rho\sigma_s - \frac{\int_o^x F_i dx}{D} = 0 \tag{36}$$

(2) Shrinkage and drop in temperature

$$\sigma_c = \frac{\sigma_s}{n} + E_c \left[Z + \Delta T(\alpha c - \alpha s)\right] \tag{37}$$

(3) Friction

$$\frac{d\sigma_c}{dx} = \frac{-F_i}{D} \quad X \quad \frac{1}{(1+pn)} \tag{38}$$

(4) Concrete movement

$$Y_c = \int_o^x \frac{\sigma_c}{E_c} dx - (Z + \alpha c\Delta T)x \tag{39}$$

(5) Joint width

$$\Delta X = x \left[\frac{\sigma_c}{E_c} - (Z + \alpha c\Delta T)\right] \tag{40}$$

Model-2:

(1) Equilibrium

$$\sigma_{co} + \rho\sigma_{so} = \rho\sigma_{sc} + \frac{\int_o^x F_i dx}{D} \tag{41}$$

(2) Shrinkage and drop in temperature

$$\sigma_c = \frac{\sigma_s}{n} + E_c[Z + \Delta T\ (\alpha s - \alpha s)] \qquad (42)$$

(3) Friction

$$\frac{d\sigma_c}{dx} = - \frac{F_i}{D} \times \frac{1}{(1+pn)} \qquad (43)$$

(4) Concrete movement

$$Y_c = \int_o^x \frac{\sigma_c}{E_c}\, dx - (Z + \alpha c \Delta T)x \qquad (44)$$

(5) Steel boundary conditions

$$\int_o^a \sigma_s dx + \int_a^b \sigma_s dx = E_s \alpha_s \times \Delta T \qquad (45)$$

SUMMARY OF MODEL STUDY RESULTS

The available pavement structure response and pavement distress models for the major distresses found in each pavement type have been discussed in detail herein. The models were selected as best suited for the prediction of the assigned distresses. Some distresses have not been assigned a specific model, because no suitable mathematical models were found for prediction of these distresses.

APPENDIX D

EXPANDED SEQUENCE OF DEVELOPMENT FOR SELECTED DISTRESSES

The basic sequence of pavement distress development discussed in Chapter 4 was applied to each of those distress types identified in Chapter 2 as having the highest priority for further study in this project. At each step in the basic sequence, an attempt was made to include as many related effects and interrelationships as possible.

Flexible Pavement Distress

The distress types for asphalt concrete pavements, that were determined to be most significant for the study of distress/performance relationships, are identified in Table 2 as (1) fatigue cracking, (2) rutting, and (3) low-temperature cracking. Detailed flow charts, illustrating the development of each of these distress types, are given in Figures 69, 70, and 71. Note that these figures are interrelated, which is an indication of the compound nature of the development of these distresses.

Composite Pavement Distress

Table 2 lists the most significant composite pavement distress variable chosen for study in this project. The highest priorities are assigned to: (1) reflection cracking, (2) fatigue cracking, and (3) rutting. Figures 72, 73, and 74 demonstrate the development of these distresses. Again, the cross-referencing which occurs in these figures indicates that the development of each distress type is interrelated to the development of the other distresses.

Rigid Pavement Distress

Table 2 lists the most significant rigid pavement distress variables chosen for study in this project. The distress variables are delineated by type of rigid pavement, i.e., jointed concrete pavement (JCP), jointed reinforced concrete pavement (JRCP), and continuously reinforced concrete pavements (CRCP). Several of the distresses are common among these types of concrete pavement, but are listed at a different priority level and therefore occur as individual entries in Table 2. The common elements have been treated within the figures contained in this section of the report and are differentiated only if there are particular features of the action of a type of pavement that requires amplification. As can be noted in Table 2, the following common types of distress variables included are: (1) faulting - joint and crack, (2) fatigue cracking, (3) spalling - joint and crack, (4) low temperature and shrinkage cracking. Other distress variables listed in the table include D-cracking, blowups and deep crack spalling.

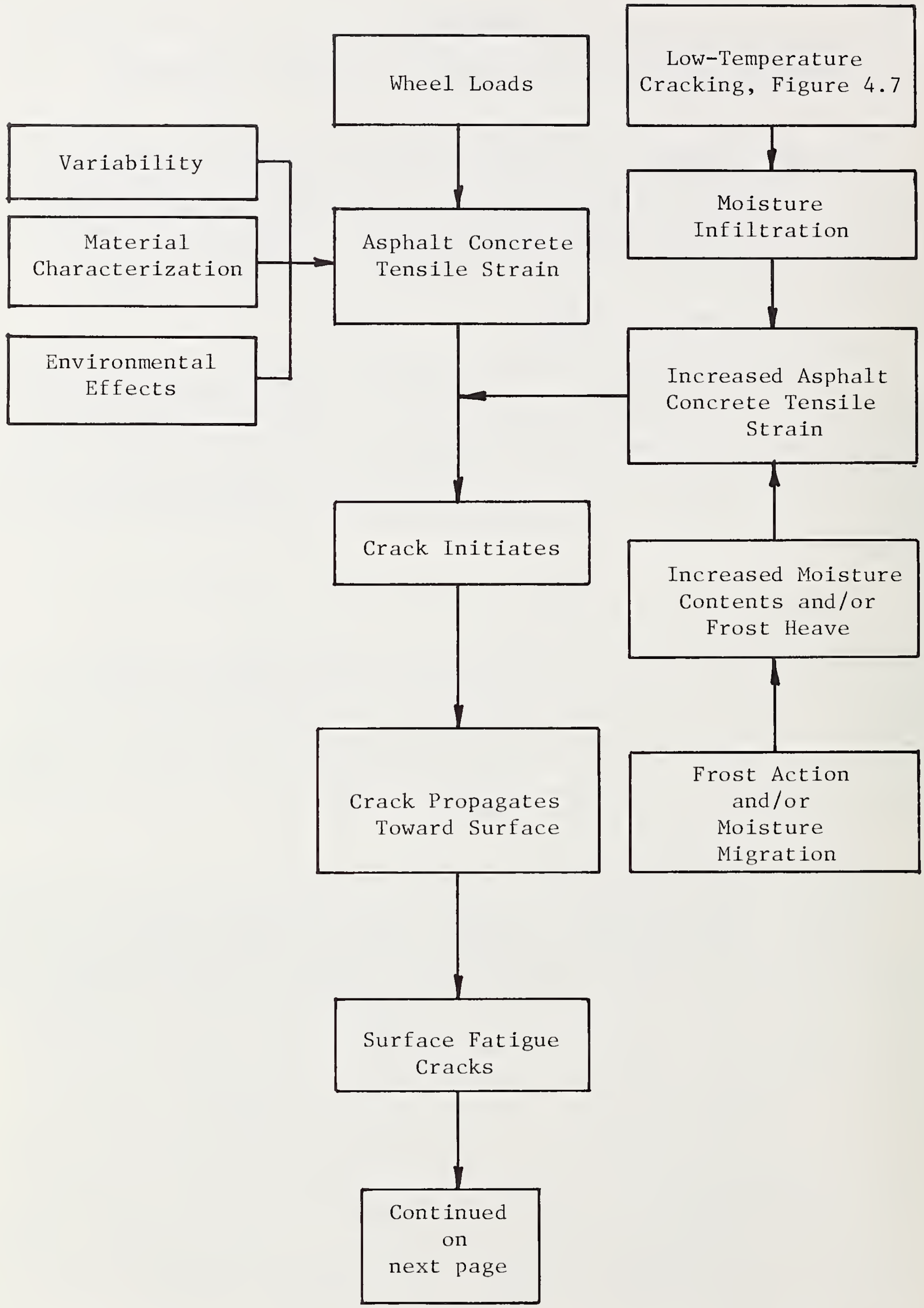


Figure 69 Fatigue cracking development in an asphalt concrete pavement structure (after Ref.59)

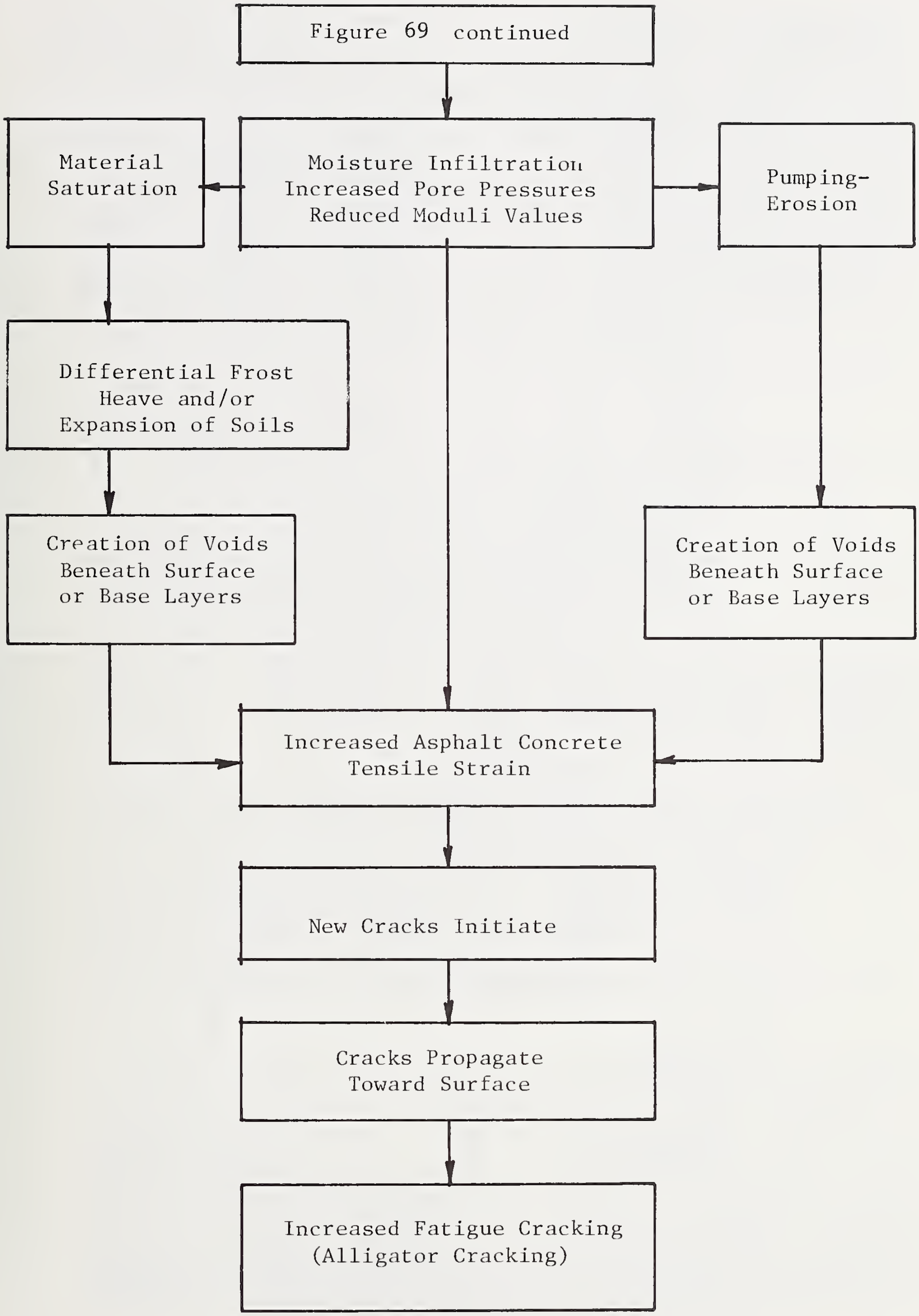


Figure 69 Fatigue cracking development in an asphalt concrete pavement structure (continued)

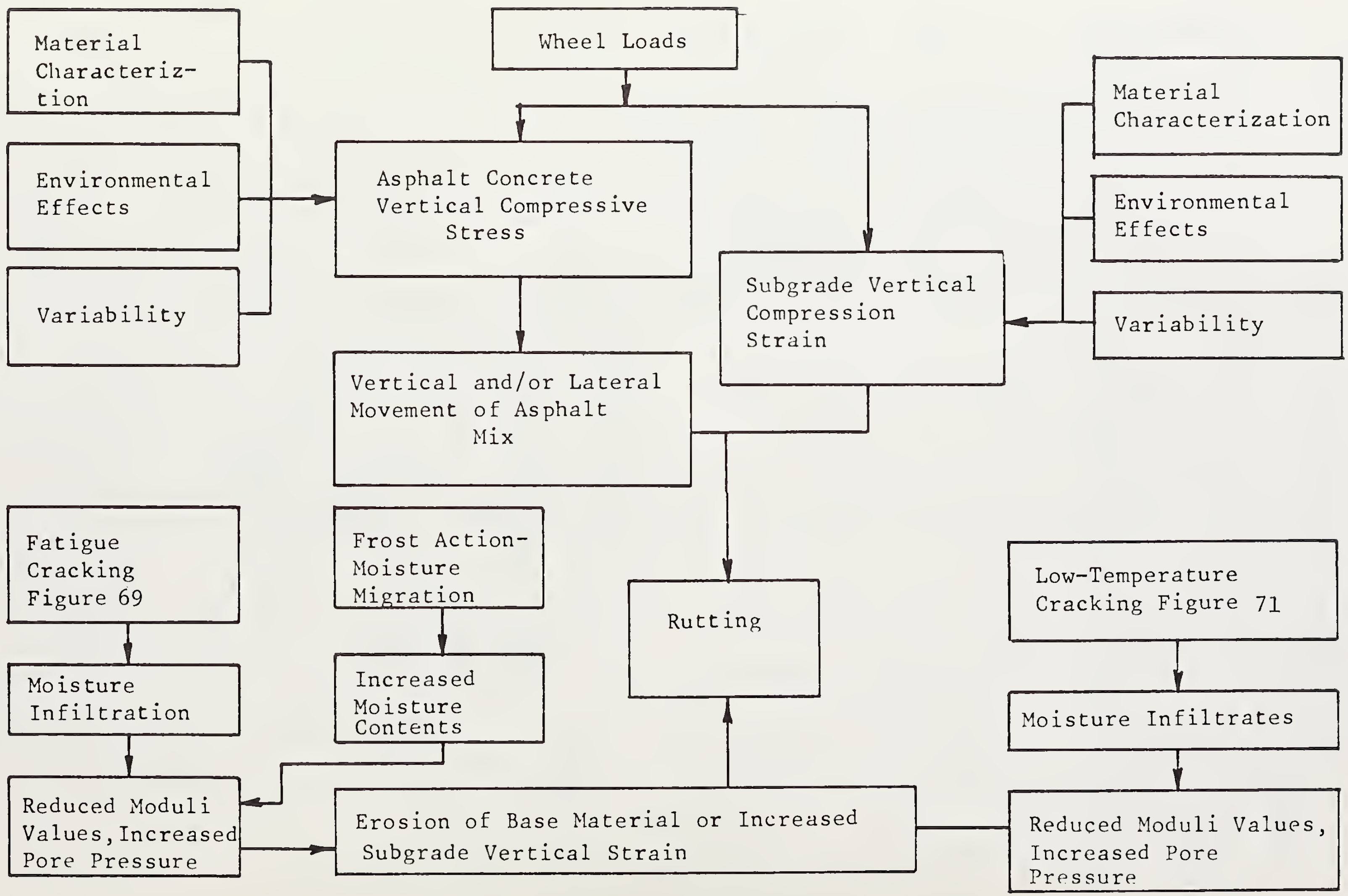


Figure 70 Rutting development in an asphalt concrete pavement structure (After Ref. 59)

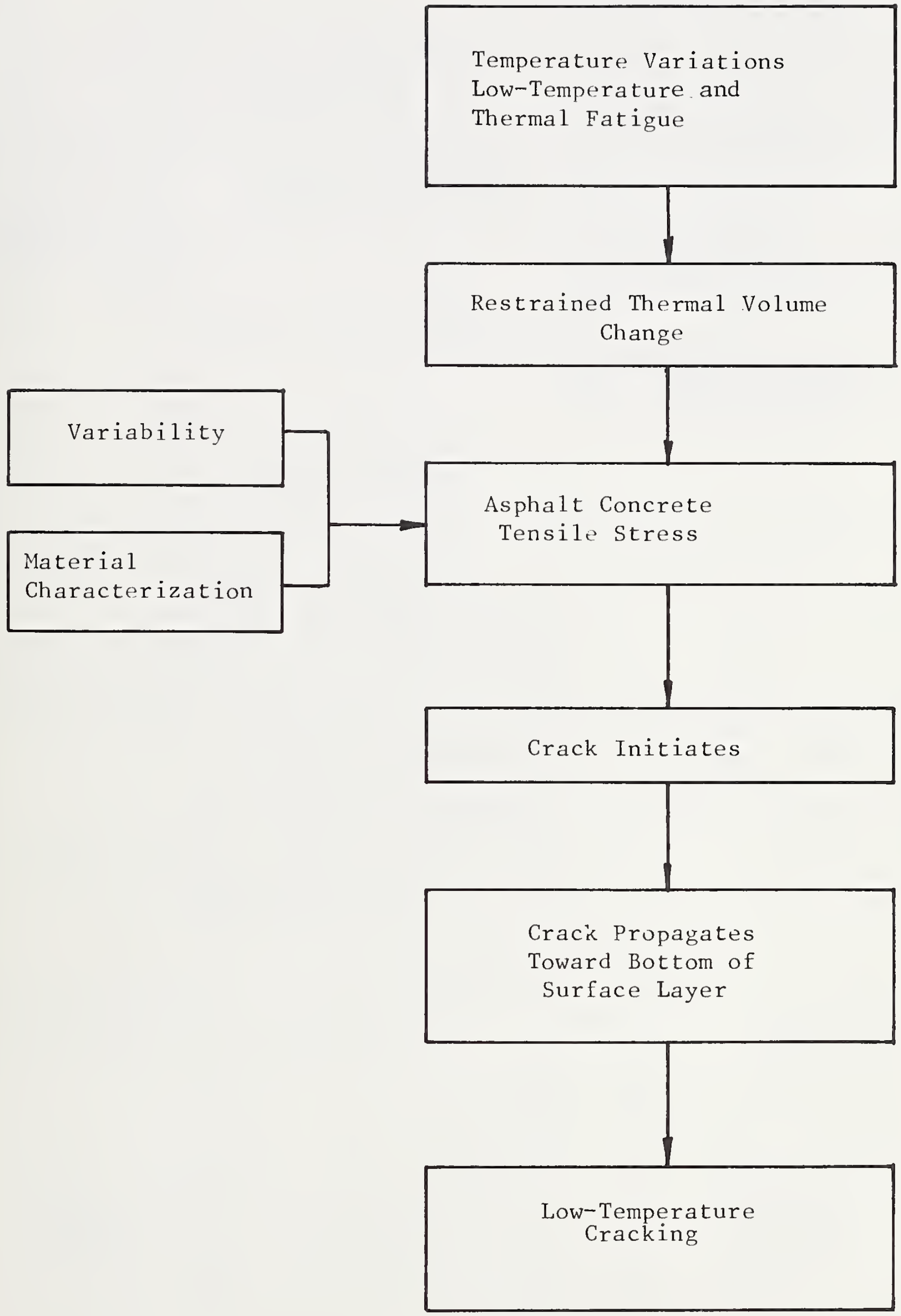


Figure 71 Low-temperature cracking development in an asphalt concrete pavement structure (After Ref.59)

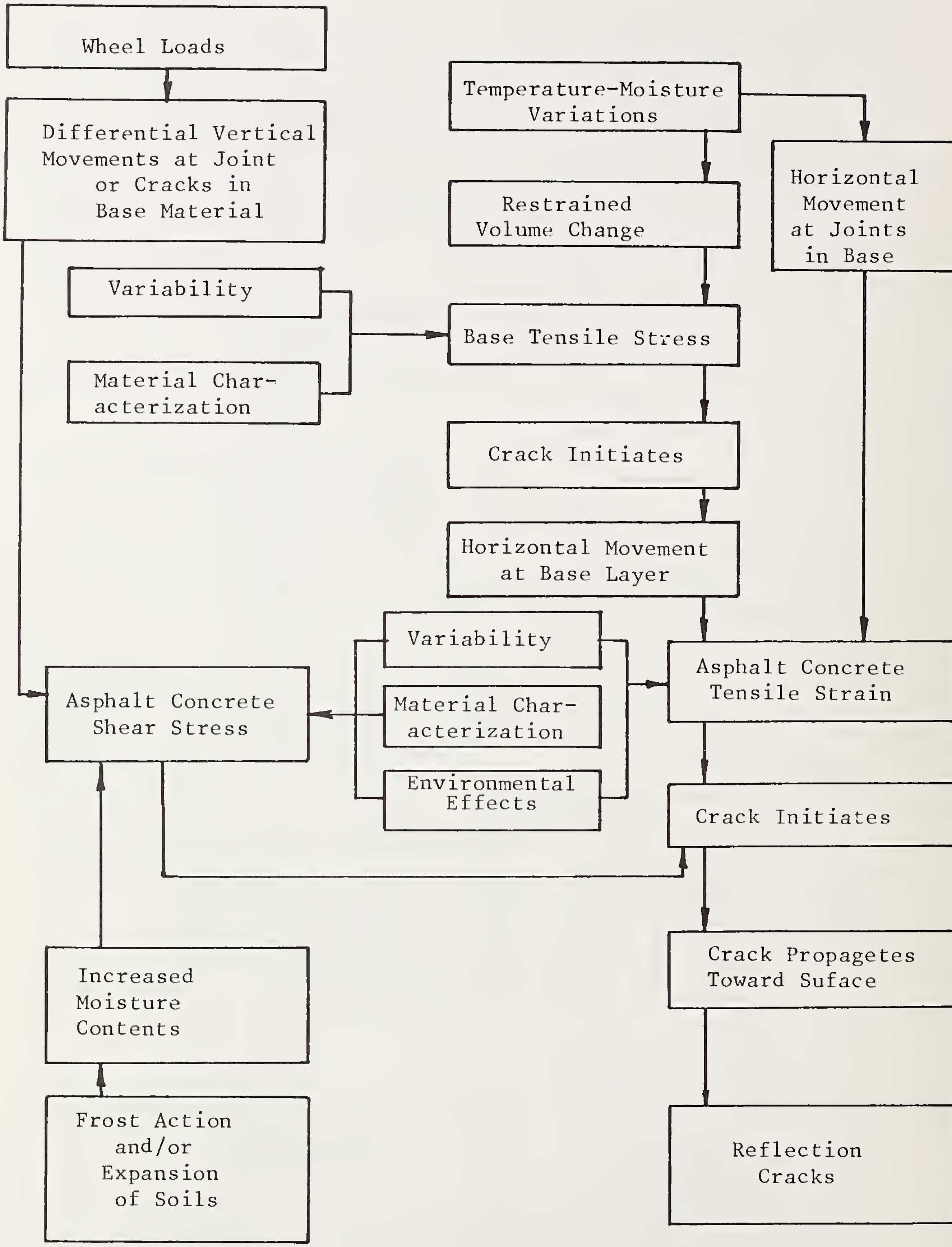


Figure 72 Reflection cracking development in a composite pavement structure (After Ref.59)

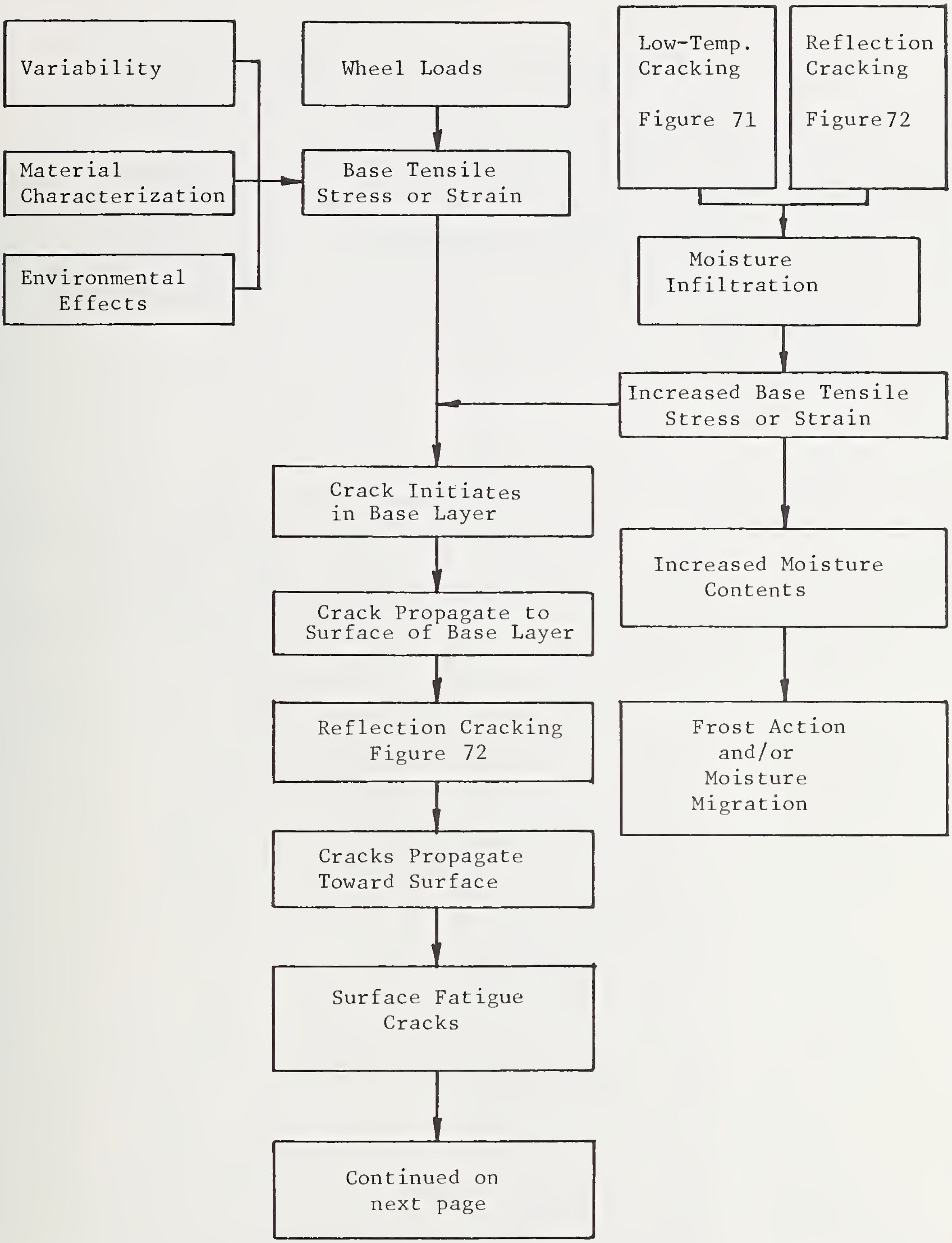


Figure 73 Fatigue cracking development in a composite pavement structure (After Ref. 59)

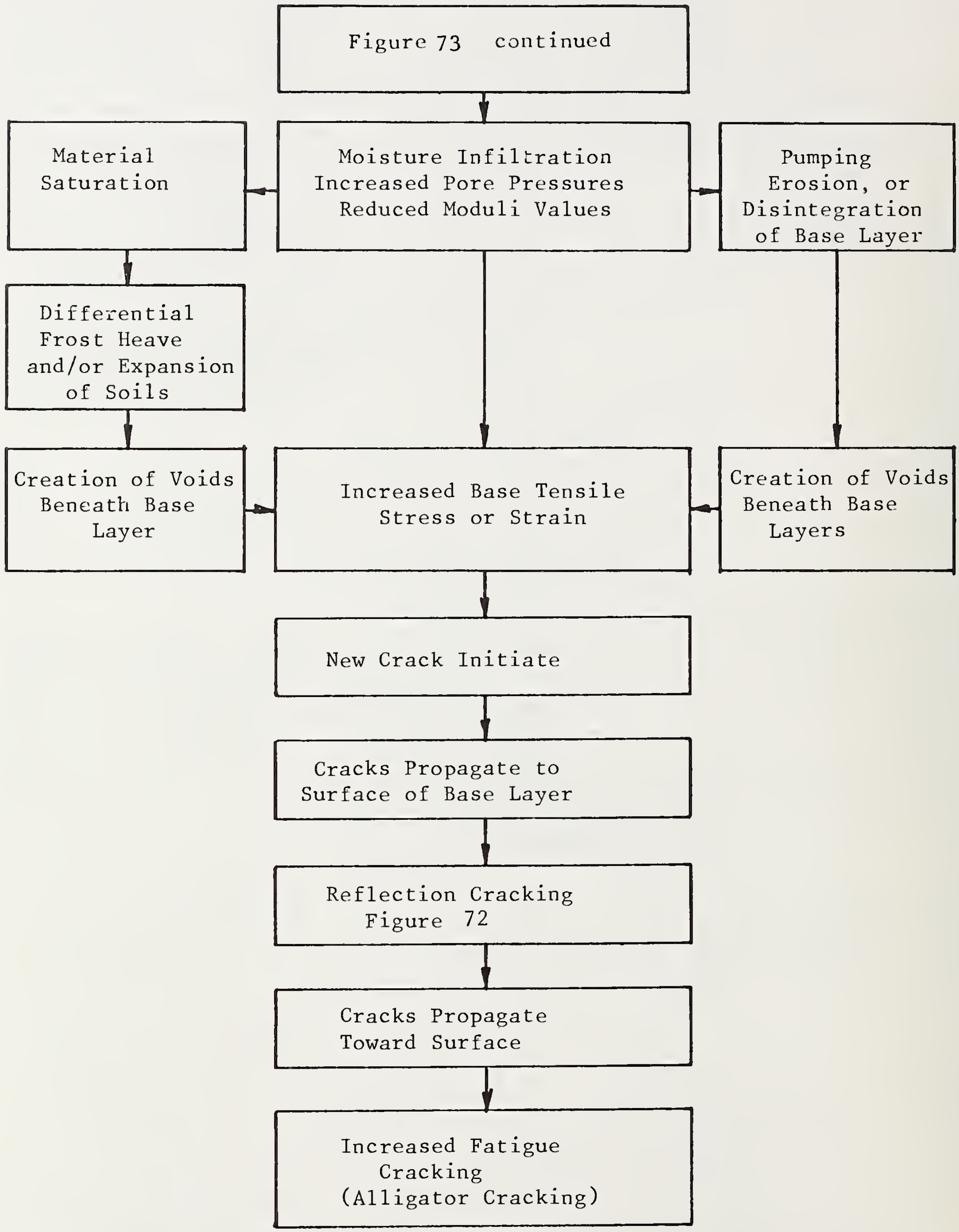


Figure 73 Fatigue cracking development in a composite pavement structure (continued)

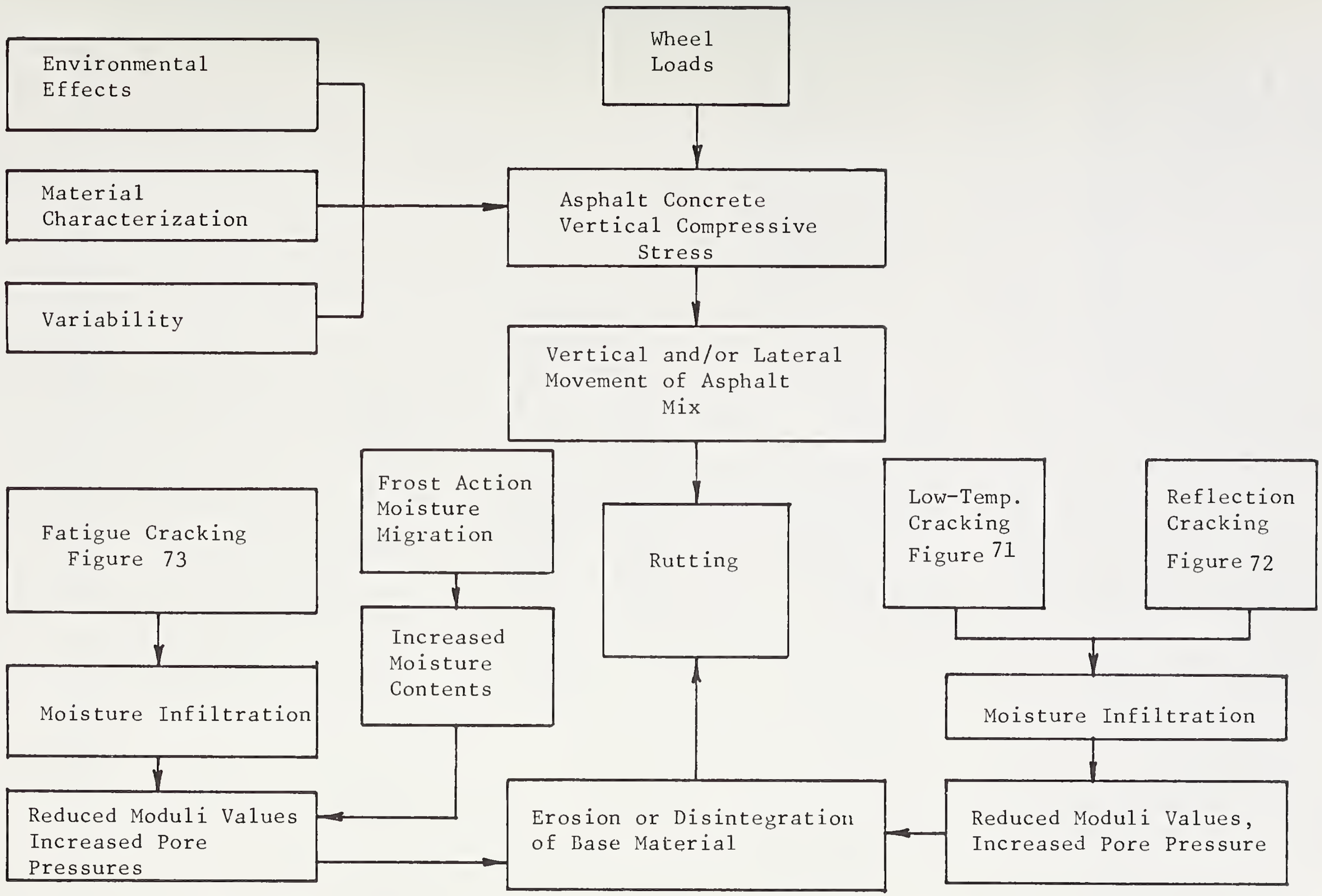


Figure 74 Rutting development in a composite pavement structure (After Ref. 59)

A series of figures have been included to indicate the general development of each of the types of distress variables included in this study. It can be noted that several of the figures utilized information contained in previous figures and are shown as references to previous figures in this chapter. The figures have been prepared to indicate the general types of factors that enter into the production or development of each of the types of distress. Figures 75 through 81 contain the flow diagrams showing the steps in the development of the various distresses in the portland cement concrete pavements considered important in this study.

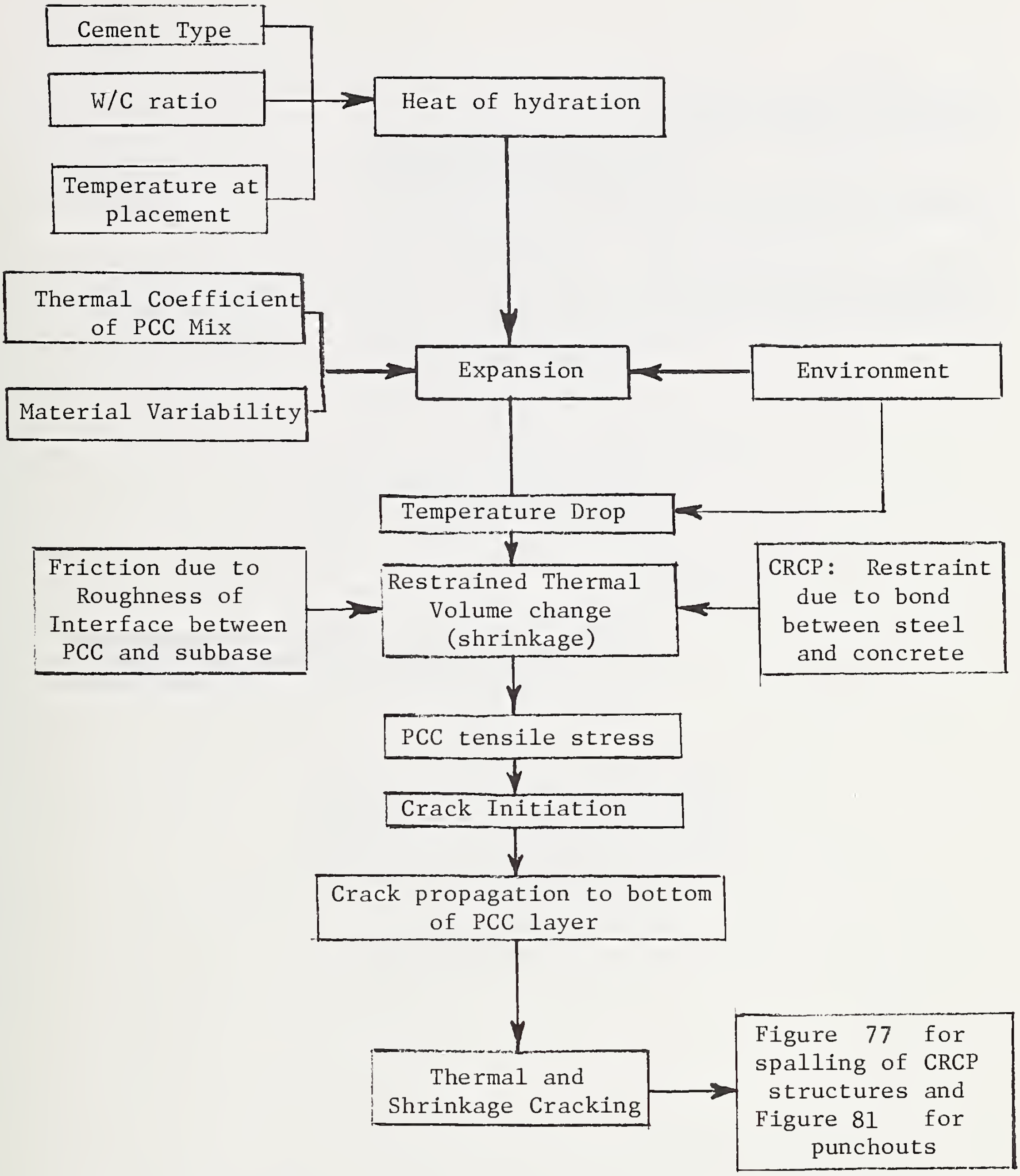


Figure 75 Low temperature and shrinkage cracking development in a PCC pavement structure.

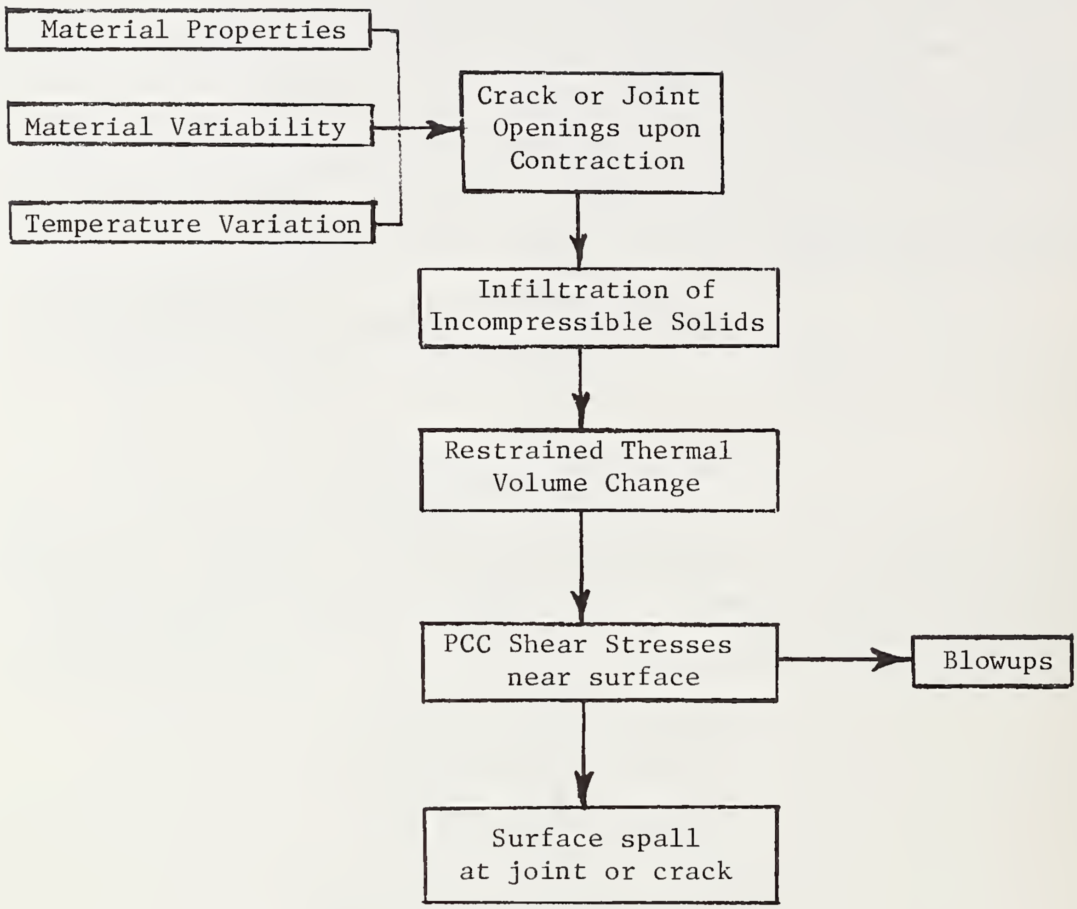


Figure 76 Development of joint and crack surface spalling in JCP and JRCP structures

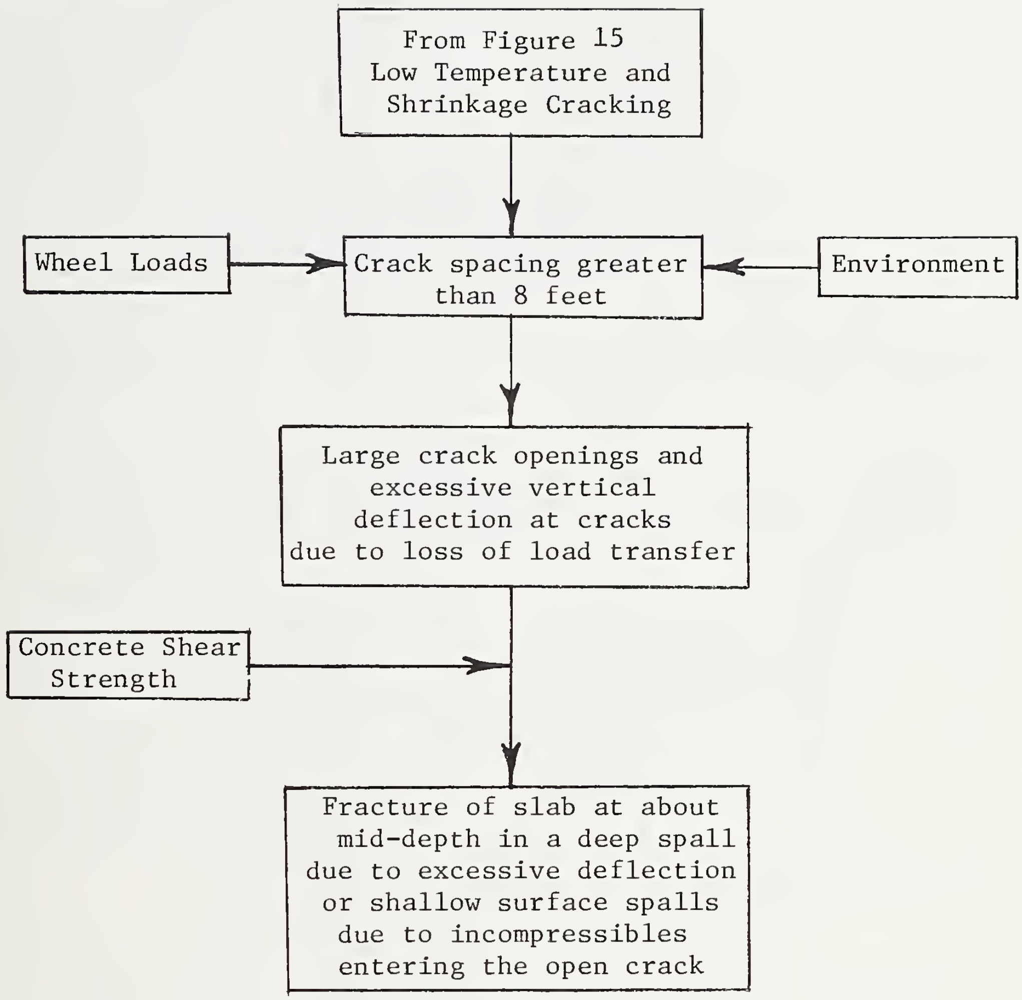


Figure 77 Development of crack spalling in CRCP structures.

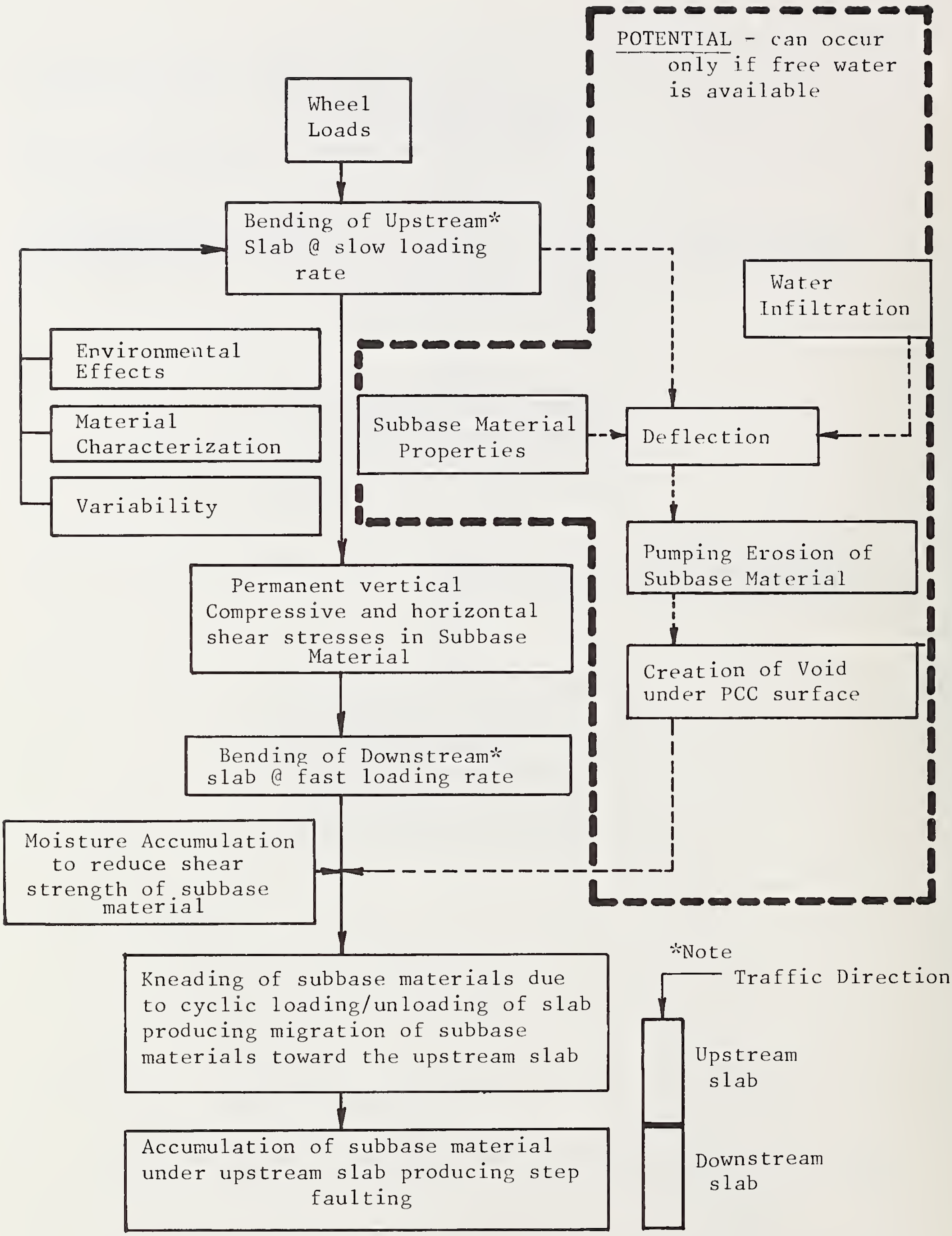


Figure 78 Development of faulting at a joint or crack in a PCC pavement structure

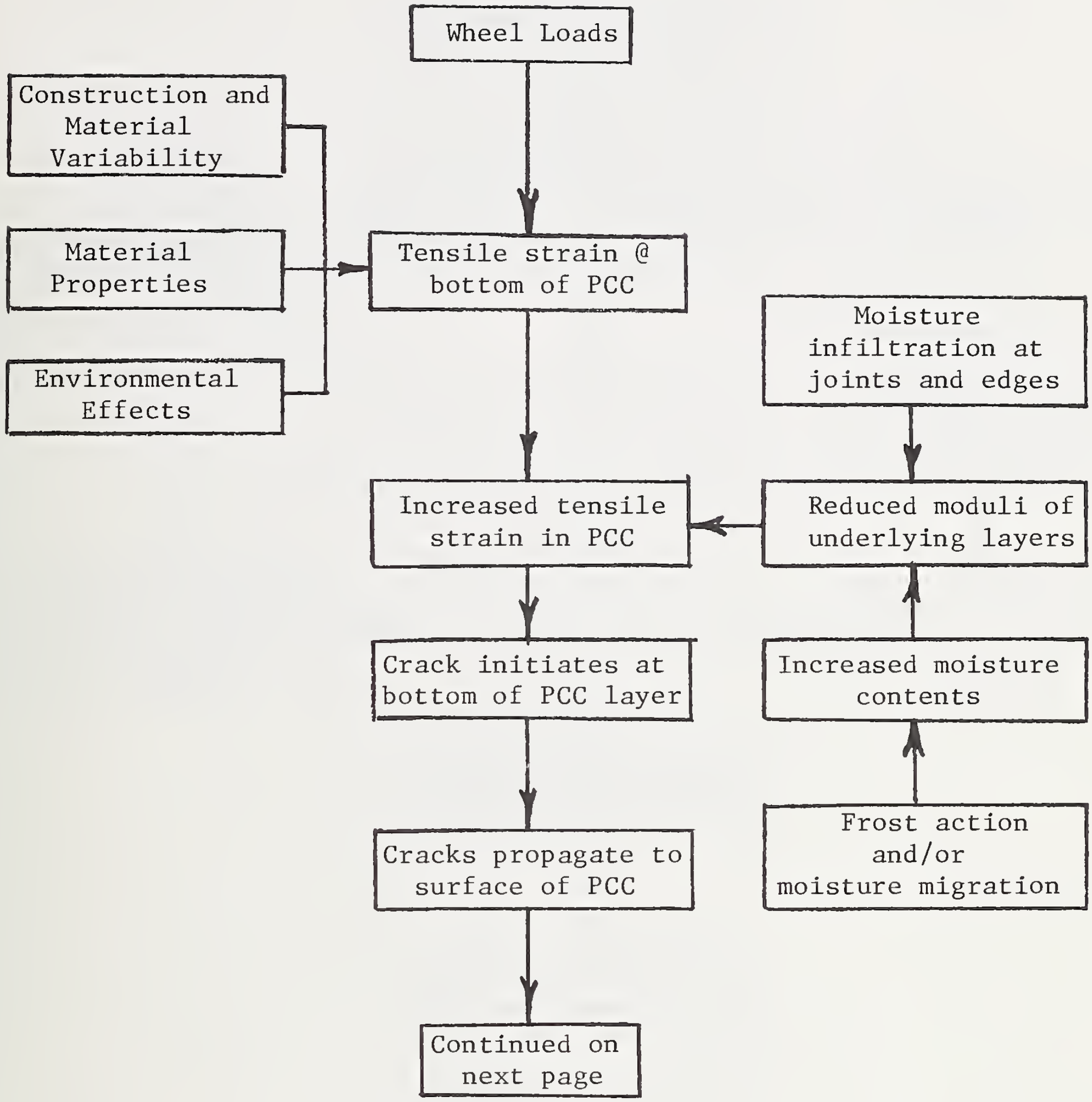


Figure 79 Fatigue cracking development in a PCC pavement structure.

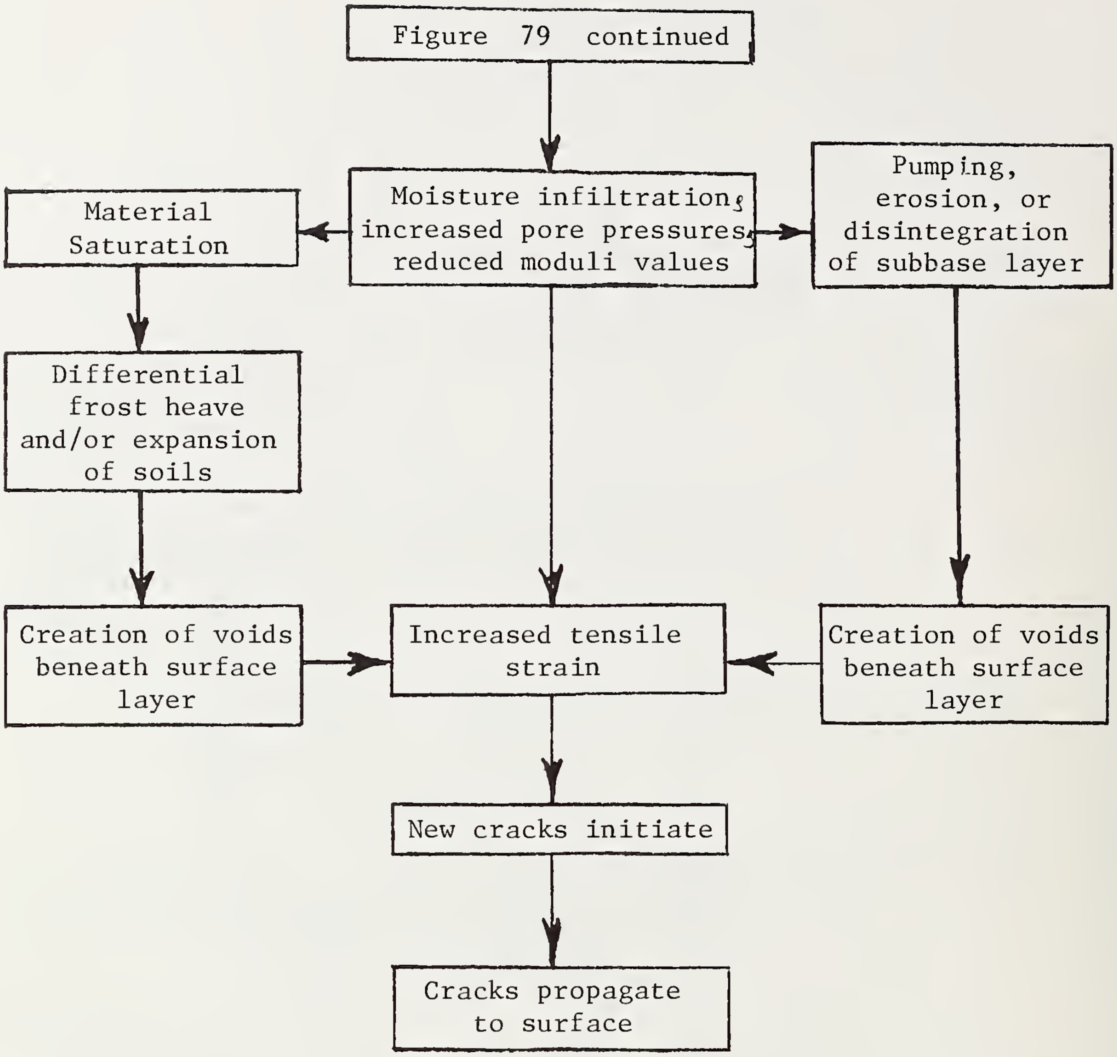


Figure 79 Fatigue cracking development in a PCC pavement structure (continued).

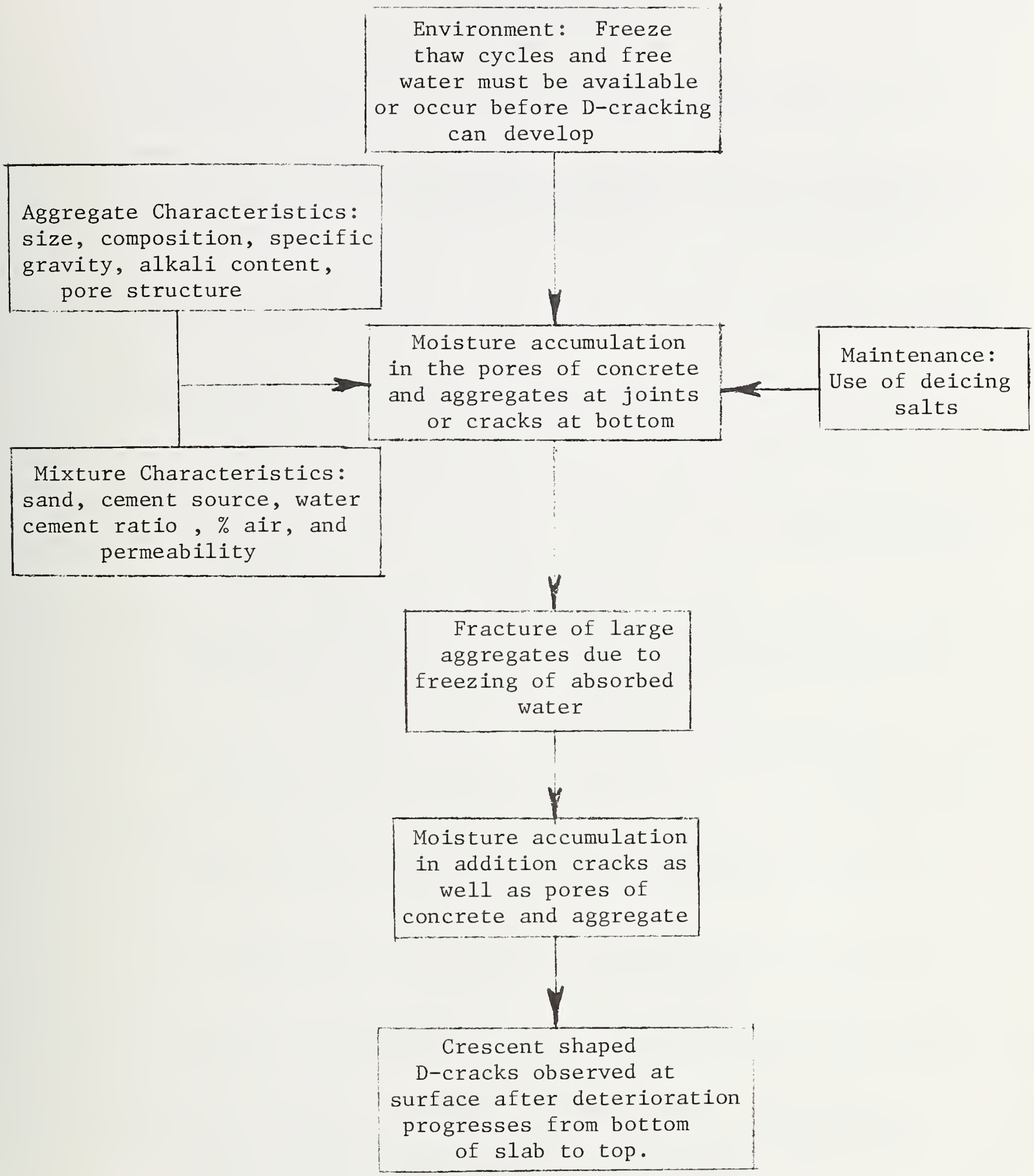


Figure 80 Development of D-cracking in a PCC pavement structure.

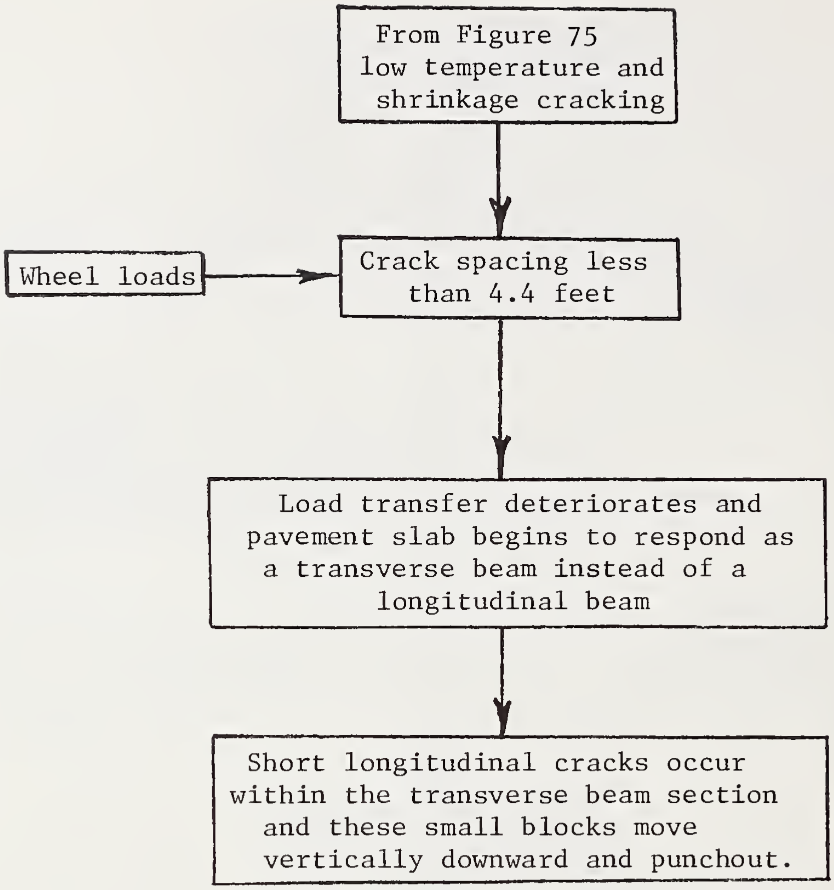


Figure 81 Development of punchouts in CRCP structures.

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The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

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